

**RISK ASSESSMENT METHODS IN
ROAD NETWORK EVALUATION:
A Study of the Impact of Natural
Hazards on the Desert Road, New
Zealand.**

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ABSTRACT

This study investigates hazards that have the potential to close the Desert Road, which is part of New Zealand's major north-south link, State Highway 1. It provides a case study for the application of risk assessment methodology to the evaluation of road networks.

The hazards that are investigated include snow and ice conditions, volcanic eruptions and lahars, seismic events, and traffic accidents. All of these hazards have the potential to close the Desert Road.

For each of the hazards, a stochastic model is developed to determine the probability of the hazard occurring and the resulting road closure duration. The vulnerability of alternative routes through the Central North Island to these hazards are also evaluated.

A traffic assignment model, SATURN, is used to predict the disruption caused by closures of the Desert Road and its alternative routes, quantifying the economic cost of closures to the New Zealand economy. Monte Carlo simulation is then used to find the probability distribution of the average annual cost of closures due to each hazard.

Mitigation options that may either reduce the probability of closure occurring, or reduce the duration of closures, are investigated. The new risk of closure with the mitigation in place is compared to the existing risk of closure, to find the probability distribution of the benefit-cost ratio for each mitigation.

A computer based risk optimisation program is described that can help select the portfolio of mitigation options that will optimise the risk reduction attained for a given expenditure.

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TABLE OF CONTENTS

1. GENERAL INTRODUCTION	9
1.1 Introduction	9
1.2 Risk Analysis Techniques	11
1.2.1 Definition of Risk	11
1.2.2 Risk Analysis	11
1.2.3 Risk Management	15
1.3 Applicability to Road Network Evaluation	15
1.4 Assessment of Closures on the Desert Road	16
1.4.1 Purpose of this Study	17
1.5 Outline of this Thesis	18
1.6 References	19
 2. CLOSURE DUE TO SNOW AND ICE	 21
2.1 Introduction	21
2.2 Modelling Techniques	21
2.2.1 Historical Frequency of Road Closure	21
2.2.2 Frequency of Causative Mechanisms	22
2.3 Snow and Ice Model Development	22
2.3.1 Definition of Road Closure Conditions	23
2.3.2 Icing Conditions and Closure Duration	25
2.3.3 Probability of Icing Conditions	32
2.3.4 Combining the Frequency and Closure Duration Relationships	34
2.3.5 Status of Alternative Routes	37
2.4 Conclusions	40
2.5 References	40
 3. CLOSURE DUE TO VOLCANIC ACTIVITY	 41
3.1 Introduction	41
3.2 Modelling Techniques	41
3.3 Volcanic Ash Model Development	43
3.3.1 Probability of Volcanic Activity	44
3.3.2 Calculation of Ash Depths following Volcanic Activity	46
3.3.3 Modelling of the Wind Environment	49
3.3.4 Closure Duration due to Volcanic Ash	59
3.4 Volcanic Lahar Model Development	63
3.4.1 Probability of Lahar Formation	63
3.4.2 Determination of Lahar Paths	65
3.4.3 Closure Duration due to Lahars	65
3.5 Probability of Closure Durations	66
3.6 Status of Alternative Routes	66
3.7 Conclusions	66
3.8 References	67

4. CLOSURE DUE TO EARTHQUAKES	71
4.1 Introduction	71
4.2 Modelling Techniques	71
4.2.1 Works Consultancy Services Limited Method	72
4.2.2 Maffei Method	73
4.2.3 Approach used in this Study	73
4.3 Earthquake Model Development	74
4.3.1 Assessment of Bridge Vulnerability	74
4.3.2 Earthquake Intensity and Closure Duration	78
4.3.3 Probability of Earthquake Intensities	81
4.3.4 Probability of Closure Durations	83
4.3.5 Status of Alternative Routes	85
4.4 Conclusions	85
4.5 References	85
5. CLOSURE DUE TO TRAFFIC ACCIDENTS	87
5.1 Introduction	87
5.2 Modelling Techniques	87
5.3 Accident Model Development	88
5.3.1 Accident Probabilities	88
5.3.2 Duration of Road Closure after an Accident	91
5.3.3 Probability of Road Closures	93
5.3.4 Status of Alternative Routes	93
5.4 Conclusions	94
5.5 References	95
6. INTERDEPENDENCY OF CLOSURE MECHANISMS	97
6.1 Introduction	97
6.2 Analysis of Relationships	98
6.2.1 Snow and Ice / Volcanic Events	98
6.2.2 Snow and Ice / Earthquakes	99
6.2.3 Snow and Ice / Traffic Accidents	99
6.2.4 Volcanic Events / Earthquakes	102
6.2.5 Volcanic Events / Traffic Accidents	103
6.2.6 Earthquakes / Traffic Accidents	104
6.3 Conclusions	104
6.4 References	104
7. ASSESSING THE RISK OF ROAD CLOSURE	107
7.1 Introduction	107
7.2 Definition of Closure Risk	107
7.3 Development of a Traffic Model for the Central North Island	108
7.3.1 Representation of the Road Network	108
7.3.2 Prediction of the Origin-Destination Matrix for Travellers on the Network	109
7.3.3 Simulating the Current Travel Regime	111

7.3.4 Confirmation of Model Reality	114
7.4 Costs of Traffic Movement	116
7.4.1 Vehicle Operating Costs	116
7.4.2 Occupant Travel Time Costs	117
7.4.3 Cost of Traffic Accidents	118
7.5 Modelling Traffic Behaviour During Road Closures	122
7.5.1 Elasticity of Travel	123
7.6 Economic Cost of Road Closures	125
7.7 Comparison of Risk Imposed by the Hazards	126
7.8 Conclusions	129
7.9 References	129
 8. ANALYSIS OF MITIGATION OPTIONS	 131
8.1 Introduction	131
8.2 Analysis Approach	131
8.3 Economic Evaluation	133
8.4 Mitigation Options	135
8.4.1 Specialist Grader Equipment	136
8.4.2 Application of Chemicals	139
8.4.3 Temporary Access Past Damaged Bridges	147
8.4.4 Seismic Retrofit of Bridges	150
8.4.5 Road Realignment for Accident Reduction	156
8.4.6 Variable Message Signs	159
8.5 Conclusions	164
8.6 References	165
 9. OPTIMUM DISTRIBUTION OF RESOURCES	 167
9.1 Introduction	167
9.2 Program Development	167
9.2.1 Optimisation Method	167
9.2.2 Calculation Details	169
9.2.3 Incorporating Step Functions	172
9.2.4 Mitigation Combinations	173
9.2.5 Program/User Interface	174
9.2.6 Program Output	174
9.2.7 Incorporating Uncertainty	175
9.2.8 Future Developments	176
9.3 Potential Applications of Methodology	177
9.4 Conclusions	177
9.5 References	178
 10. CONCLUSIONS	 179
10.1 Use of Risk Assessment Methodology in Road Network Evaluation	179
10.2 Cost of Road Closures to the New Zealand economy	180

10.3 Mitigation Strategies for the Desert Road	181
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Appendices

A. Mean Intervals between Volcanic Eruptions	185
B. Hazard Zone Maps for Mt Ruapehu	187
C. Closure durations along Alternate Routes due to Lahars formed at Ruapehu	193
D. Seismic Vulnerability Calculations for Desert Road Bridges	195
E. Vulnerability Rating of Bridges along Alternative Routes	203
F. Schematic Representation of the Buffer Network Used to Model the Central North Island	205
G. Proposed Waikato Stream to Wharepu Realignment	207
H. User Interface for Risk Optimisation Software	209

1. GENERAL INTRODUCTION

1.1 Introduction

Risk assessment is a systematic process that can be used for describing and quantifying the risks associated with hazardous substances, processes, actions, or events. It allows an analyst to incorporate uncertainties, such as the value and variability of input parameters and the model limitations, into the decision making process. This produces a probability distribution of the expected outcome as opposed to a point estimate, allowing decision makers to interpret the information with an explicit knowledge of the uncertainty of the result. It also enables the sensitivity of the system response to different input variables to be evaluated.

Worldwide there is a growing awareness of the vulnerability of transportation networks, such as roads, to damage by natural hazards. This damage can incur massive financial costs for repair, and cost local economies through disruption caused by subsequent road closures. Although some natural hazards are relatively rare events, these costs are significant, and should be considered when evaluating the benefits of preventative road improvements. Butcher in 1985 showed that such a risk assessment would be a valuable addition to current road evaluation procedures in New Zealand. He used the Arthur's Pass route as a case study for the use of risk analysis in the economic evaluation of roading improvements. Other projects that have used risk analysis in project evaluation include the Hawkes Bay Motorway Bridge, Newlands Interchange, and the Thorndon Overbridge. To extend this work, the applicability of risk analysis to the roading network needs to be investigated further.

With recent publicity and a high public sentiment over road closures, the Desert Road in the Central North Island, New Zealand (Figure 1.1) provides an ideal case study into the use of risk assessment methods. Limited alternative routes for travel when the Desert Road is closed allows the impacts of closure on traffic flow in the surrounding road network system to be assessed more easily, with a limited number of entry and exit points to the system. In addition, the Desert Road is a relatively long stretch of road (53km) that has no significant entry or exit points along its length. This means that if any one part of the Desert Road is closed, the whole of the road link is closed.

Hazards causing the Desert Road to close are invariably of low probability, and consequently there is limited historical data available. Because of this lack of information, there is a high degree of uncertainty in estimates of road closure probabilities. This is a characteristic dealt with effectively by risk assessment methods. In recent years the Desert Road has been frequently closed because of snow and ice on the road. During one of these extended closures (a 9 day closure), measurements were taken of the impacts on traffic flow on alternative routes. This provides some initial information on the behaviour of drivers travelling in this area during road closures.

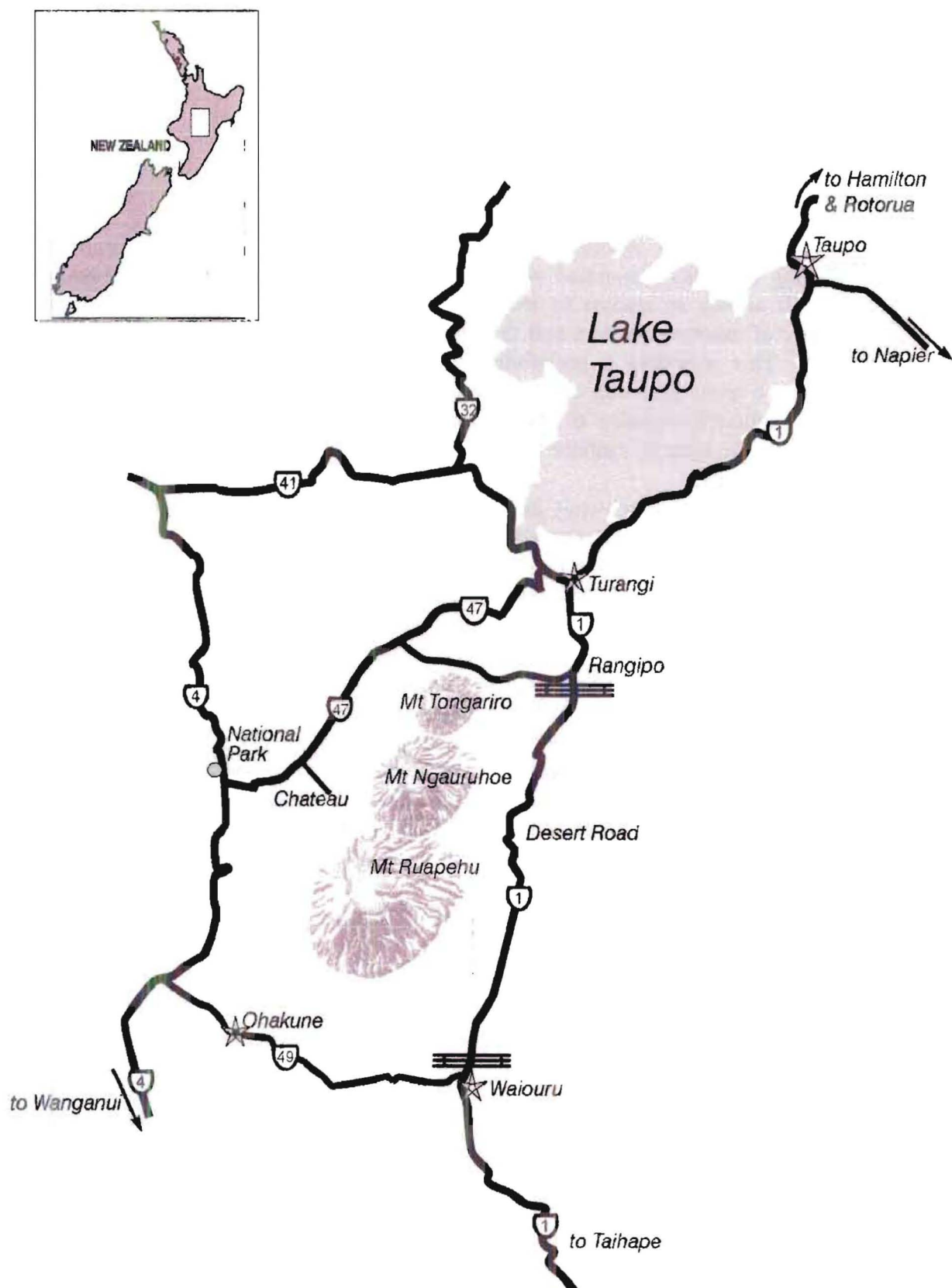


Figure 1.1: Location Map for the Central North Island, New Zealand

The objectives of this study are to look at the potential for environmental factors such as natural hazards to close the Desert Road, and establish probabilities with which this is likely to occur. This is done by modelling the probability and severity of road closure that could result from each natural hazard event. These models are developed in @RISK (Palisade Corporation, 1997), an Excel add-in program, which allows the effects of uncertainty in the model inputs to be assessed. To estimate the consequences of resulting road closures in terms of the monetary cost to the New Zealand economy, the response of drivers to the altered road network is then assessed using SATURN (Van Vliet, 1995), a traffic assignment software. This allows the annual cost of road closures due to a particular natural hazard to be assessed. This information is then used to evaluate the merits of various mitigation options that are aimed at reducing the risk of road closure. A risk balancing software is developed that helps determine the optimal spending portfolio to reduce a maximum amount of risk within a fixed budget.

1.2 Risk Analysis Techniques

1.2.1 Definition of Risk

Risk may be defined as the combined effect of the probability of occurrence of an undesirable event, and the magnitude of the event (IPENZ, 1983). Risk is typically quantified as a function of both the likelihood and the consequences of the undesirable event, usually with the product of these factors equalling the level of risk (Smith, 1993);

$$R = pL \quad (1-1)$$

where p is the probability of the event occurring, and L is the loss, or consequences of the event.

A third factor however also impinges upon the definition of risk, and that is the context of the risk. The context sets the frame of reference of the problem. It is the decision to be taken, and who is making the decision. It gives the analysis its bounds, reasons, and purpose, without which the assessment is likely to be of little use.

1.2.2 Risk Analysis

In making any decision, whether formally or informally, some kind of risk analysis is undertaken. When making a decision, the possible consequences of an action are usually evaluated.

“Risk analysis is a vehicle for examining the data surrounding a decision problem in the light of all the pervasive uncertainties of the world” (Hertz, 1983).

Risk analysis involves three main steps; that of hazard identification, risk assessment and risk evaluation.

- **Hazard Identification**

One of the main tasks in any risk assessment is to learn about the problem. This requires that the analyst finds out about the process being analysed, understands the context of the problem and its constraints, and what could go wrong (Elms, 1992b).

Hazard (or risk) identification involves exploring the problem, discerning possible scenarios, and the conditions required for their existence, where risk is imposed. Several people may be involved in this process, with fresh approaches and insights to the problem being helpful. It is important to involve people with a wide range of expertise to get a thorough coverage of all possible hazards. Once a variety of risk sources are identified, a weeding process can begin to extract those sources likely to pose significant risk.

- **Risk Assessment**

Risk assessment can be defined as "the integrated analysis of the risks inherent in a product, system, or facility, and their significance in an appropriate context" (IPENZ, 1983).

Approaches to developing a risk assessment method will be as many and varied as the intended purpose of the risk model. Petak (1982) developed a generalised procedure for use in determining the risks posed by natural hazards. This process, illustrated in Figure 1-2, provides guidance of how the governing parameters combine to determine the level of risk posed. Guidelines such as these provide a basis upon which to approach a risk assessment. The individual characteristics of the problem, availability of input data, and the intended final use of the assessment however will strongly influence the final approach taken. Information that is required from the model output will influence the form and underlying units in which total risk is quantified. Working backwards from the model output, the units used to define the probability and consequence measures can be determined. Ways to establish these measures of probability and consequence will be dependent upon the availability of data. For example, if enough data are available, the probability of an event may be calculated directly from observation. In many instances however this is not possible, and probability estimates need to be established through modification and adaptation of the data.

Uncertainties

Risk estimation is a process of prediction, and as such is unlikely to be precise. The uncertainty of a parameter will be a function of the quality of both the observed data, and the relevance and applicability of this data to its proposed use.

Sources of uncertainty in parameter estimates include:

- random errors and statistical variation such as occur in the measurement of a parameter value;
- systematic errors as may be caused by biases in the measuring equipment;
- variability in parameter values over time and space;

- general inherent randomness in the actual parameter value;
- and disagreement between expert opinions of a parameter value.

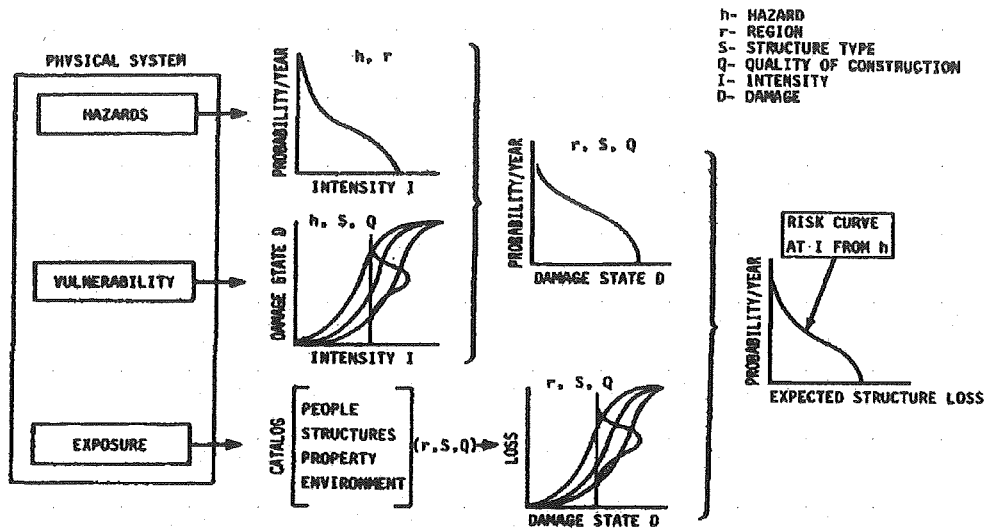


Figure 1-2: Generalised natural hazard risk determination procedure (Petak, 1982)

Approximation uncertainties arise as the models used are by necessity only simplified versions of the real-world system being modelled. There may also be uncertainty as to whether or not the model form being used is appropriate (Morgan and Henrion, 1990). Finally, there will always be the uncertainty of the unknown hindering the completeness of any model, such as failure mechanisms that have not been identified.

Risk assessment methods allow model parameters to be input as distributions, expressing explicitly the uncertainties there are about the actual value of the parameter. Uncertainty in the input parameters will ultimately be propagated through the model, resulting in uncertainty about the estimated magnitude of risk. This uncertainty is again expressed as a probability distribution of risk.

Expressing the magnitude of risk as a probability distribution provides much more information to the decision maker than does a traditional point estimate of expected risk. Even if the expected (i.e. mean) outcome of an action is positive (desirable), there may still be a possibility of a negative result, such as financial loss. Expressing the outcome as a distribution indicates to the decision maker the relative likelihood of this negative outcome. Depending upon their risk aversity, the decision maker can then decide whether or not this probability of negative consequences is acceptable.

Sensitivity Analysis

Within a model, some input parameters will have more effect upon the final model output than others. For the general relationship $y = kx^r$, the sensitivity factor of the model to a parameter may be described as:

$$\gamma_i = \frac{x_i}{y} \frac{\partial y}{\partial x_i} \quad (1-2)$$

where γ_i is the percentage increase in y , the model output, due to a one percent increase in x_i , the input parameter (Elms, 1985).

Uncertainty in the input parameters to which the model result is highly sensitive will greatly increase the total uncertainty in the final risk estimate. Performing a sensitivity analysis on the model highlights the input parameters for which it is most productive to reduce uncertainty.

The “principle of consistent crudeness” (Elms, 1992a) requires that there be consistency between the level of quality of the input parameters, modified by the sensitivity and the precision of the model. The level of detail of a model should essentially be governed by the crudest part of that model. There is no point in having a highly detailed model producing accurate results for a first parameter, if you are then combining these results with a highly uncertain second parameter to which the model is sensitive. It would be a more productive use of time, effort, and money to have used a more crude model for the first parameter, and then spend the extra effort improving estimates of the second parameter.

Model predictions can be improved iteratively, refining the model and its inputs according to the effect they have upon the final outcome. The eventual accuracy that needs to be attained by the risk assessment process will be dependent upon its intended purpose. Some models will require great accuracy, and yet others will only need estimates to within an order of magnitude.

• Risk Evaluation

Risk evaluation involves looking at the levels of risk quantified by a risk assessment model, and deciding whether that level of risk is acceptable. The outcome of this decision will be dependent upon the degree of risk aversity with which the risk is being judged. A risk averse person will avoid taking risks that may seem acceptable to others with less risk aversity.

The level of risk aversity assumed will be dependent upon the person or organisation making the decision. It is also affected by the type of risk being judged. A person is generally willing to accept much more risk when it is taken on voluntarily than a risk that has been forcibly imposed. For instance, a person may choose to take up sky diving as a sport, and yet finds lesser risks imposed by a neighbouring polluter intolerable. Some risks may also have a “horror” factor involved, where although the consequences may be the same (i.e. death), the type of death is viewed as undesirable. This is clearly illustrated by a public that are highly resistant to nuclear power, and yet are seemingly complacent about car accidents that impose consistently high death tolls. This may also be a function of people’s inherent fear of

the unknown. A full list of factors affecting safety judgements and levels of risk aversity may be found in Lowrence (1976).

1.2.3 Risk Management

Risk management involves determining how best risk may be reduced to an acceptable level, or eliminated. With limited resources available for risk reduction, and often several means by which this can be achieved, methods are required for prioritising mitigation procedures.

Initial efforts to lower risk may produce greater system reliability and safety for little cost. Successive improvements however will cost more to achieve the same degree of improved safety. This is commonly referred to as the “law of diminishing returns”. Usually, reducing the last residual amounts of risk are the hardest. Attaining a no-risk objective is usually extremely costly and generally impracticable.

As well as highlighting areas propagating large amounts of uncertainty, sensitivity analysis will also indicate parameters that contribute significantly to the total level of risk imposed. By concentrating efforts in reducing these parameters (or increasing if there is an inverse relationship), it is possible to optimise value for money in terms of risk reduction. The way the values of these input parameters are reduced will be dependent upon the type of risks posed. For example, if the vulnerability of a bridge to earthquakes is an important model parameter, then a mitigation strategy may be to seismically retrofit the bridge. The cost of this would need to be balanced against the cost of reducing another model parameter which may be cheaper, but to which the model is less sensitive.

1.3 Applicability to Road Network Evaluation

Traditional roading project evaluations in New Zealand prioritise projects using point estimates of the “most likely” outcome. Usually this analysis is supplemented with a sensitivity analysis. These sensitivity analyses provide an indication of the relative importance of the different factors affecting the outcome, but give no indication of the likelihood of their occurrence (Butcher, 1985). By using risk assessment techniques to find the probability of each event consequence, and from that, establishing the probability of the project attaining each benefit-cost ratio, this additional information is provided.

Problems associated with the traditional use of point estimates of parameter values, as highlighted by Jackson and Linard (1980) include:

- they give no information on the likelihood of failure;
- there is no information as to whether differences between expected outcomes for different options are statistically significant;
- there is no check against consistent errors of bias;
- where there are inputs from multiple sources it is difficult to ensure consistent treatment of uncertainty. There is a danger of multiple conservative adjustments

being made to the same data sets, as successive analysts build in their own safety factors;

- the decision maker, usually being remote from the analysis, has no “feel” for the accuracy and significance of the data. The decision maker is therefore not in a position to interpret sensitivity tests or the analyst’s qualifications to the results.

The primary advantage of performing probabilistic benefit-cost analysis (PBCA) over traditional deterministic benefit-cost analysis, is that PBCA’s provide far more information to the decision maker about the risks of the project. Greater understanding can only help to improve the quality of decisions made. This conclusion is reflected in recommendations made by Travers Morgan in a report produced for Transit New Zealand in 1992:

“Although risk analysis procedures have not been proposed for inclusion in the revised PEM Appendix A9, a trial of such procedures should be undertaken. If the trial is successful, risk analysis procedures could be introduced subsequently as a standard for larger projects.”

The increased effort required is commonly cited as a reason for not conducting PBCA’s for many project evaluations. This rationale is valid for smaller projects, or where project uncertainties are minimal. For large, complex projects however, the improved information available to the decision maker is likely to justify the costs of the analysis. In situations where little environmental or historical data is available from which to establish parameter distributions, the effort required to complete a PBCA is inflated. It is these projects however that especially require probabilistic analysis, so that the level of uncertainty and lack of knowledge are explicitly implied to the decision maker.

Comparability of event consequences can prove to be difficult, especially when it is not feasible to quantify each consequence in common units. How does one compare the loss of an ecologically sensitive area to the saving of travel time? The difficulties in weighing up consequences are also reflected in the complexity involved in judging the acceptability of risks. Methodologies and techniques that can be used to help overcome these comparison difficulties should be investigated further.

1.4 Assessment of Closures on the Desert Road

This study looks at the potential for environmental factors such as natural hazards to close the Desert Road, and establishes the probabilities with which this is likely to occur. An understanding is sought into the generating mechanisms of these hazards, establishing ways to find both the frequency of occurrence and potential consequences of the events. A systems model is developed using this information, to predict the total risk of closure of the road.

1.4.1 Purpose of this Study

Any risk assessment is driven primarily by its purpose, the question it hopes to answer, and the information it wishes to provide. Therefore it is very important with any risk assessment to clearly state the objective. Only by comparing the end product with these objectives can the success of the assessment be judged.

- **Assessing the use of risk assessment techniques in road network evaluation**

Traffic behaviour is dynamic, with traffic characteristics on alternative routes being interrelated. Using Monte Carlo simulation to establish the overall risk will allow for this dependence to be incorporated into the analysis.

This model will provide an indication of the input data requirements when using PBCA's in road project evaluations, and the application of model outputs. This will be useful for an analyst when deciding whether or not a project warrants a full probabilistic analysis. It is hoped that the models described here will also provide a guide for others performing similar analyses.

- **Comparison of risks posed by different closure mechanisms**

The risk of each hazard can be compared by converting their closure duration/probability relationships to a distribution of the possible costs incurred due to closures. This will indicate which hazards are posing the most significant risk, so that efforts can be concentrated on mitigating these. Since the relationships defining the total costs are both complex and interrelated, it is often not obvious which hazards are significant without detailed analysis. It is sometimes easier to limit risks from events that happen frequently, ignoring some of the lower probability yet higher consequence events. This especially happens when such an event has not occurred in recent memory. Risk assessment provides a rational basis upon which to rank the risks faced.

The integrity of the alternative routes when the Desert Road is closed will be dependent upon the reason for the Desert Road closure. Correlations will exist between closure of the two roads, particularly for an event such as a volcanic eruption, where if the Desert Road is affected, it is likely that the alternative routes will be similarly affected. The effect of these alternative routes being closed concurrently is likely to be significant in terms of the costs imposed to the national economy. Longer detours will be required, communities may be isolated, and many trips may be cancelled. The effect of this interdependence is not immediately obvious without detailed analysis. This study provides some insight into how these relationships work, and what effect this has upon the ranking of the risks posed by the hazard events.

- **Benefit-cost analysis of hazard mitigation**

Once a risk assessment has highlighted the hazards causing significant risks, it is then necessary to decide if these risks are acceptable. If it is found that the risk levels are too high and need to be reduced, mitigation strategies must be investigated.

Any mitigation effort will invariably have a cost associated with it. The question then becomes one of whether or not the benefit-cost ratio of the mitigation indicates its viability. This will be a function of two factors:

- the cost of the mitigation;
- and by how much the mitigation will reduce the total risk.

For example, a mitigation option to alleviate closures due to snow and ice would be the application of chemicals to the pavement surface. These chemicals effectively lower the freezing point of water, preventing ice from forming. Many mitigation options will not always be 100% effective, so there may still be times when the road will close. The economic cost of these residual closures, added to the cost of implementing the mitigation, is then compared to the cost of the road closures without the mitigation in place. This provides an estimate of the benefit-cost ratio for that mitigation option.

The analysis should also highlight within a hazard type, the event magnitudes that are causing the most risk. As was discussed earlier, the risk imposed by a hazard will be a function of both the event probability and its consequences. There will be an event magnitude that causes maximum risk. Mitigation measures aimed at reducing the probability or consequences of event magnitudes at or near this optimum will have greater effectiveness.

- **Assessing the value of information**

The beauty of using probabilistic risk assessment methods over traditional cost benefit analysis is that uncertainties in the analysis can be quantitatively assessed. Especially when dealing with natural events, the amount of data available to calibrate models can be limited. This may be coupled with a limited understanding of system interactions. Two remedial measures may have a similar expected benefit-cost ratio, yet there may be far more uncertainty associated with the effectiveness of one proposal. Expressing the benefit-cost ratio in terms of probabilities communicates this to the decision maker, and allows for the magnitude of the uncertainty to be accounted for in any decision.

If more is known and understood about a problem, any outcome predictions will have a greater reliability. The gathering of data however should be prioritised, as reducing uncertainty in some parts of the model will have more effect on the total model accuracy than others. The areas that would benefit most from increased accuracy can be identified via sensitivity analysis.

1.5 Outline of this Thesis

The first section of this thesis describes the risk models that were developed to describe the potential for environmental hazards to close the Desert Road. The hazards investigated are snow and ice (Chapter 2), volcanic eruptions and lahars (Chapter 3), earthquakes (Chapter 4), and traffic accidents (Chapter 5). These chapters describe the way in which the frequency of occurrence and potential

consequences of these hazards were established. The concurrent effects of the hazards upon the integrity of the alternative routes during Desert Road closures are also investigated. Chapter 6 investigates any interdependencies between the occurrence and effects of the different hazards.

The expected cost of road closure will increase with closure duration, though the relationship between the two is likely to be complex. Chapter 7 details the traffic model used to predict the disruption to trips caused by road closures, and the total cost to the New Zealand economy of these traffic disruptions. These costs are then combined with the hazard model results to establish the total risk of road closure that currently exists for the Desert Road.

Possible mitigation options to reduce the risk of road closure are investigated in Chapter 8. The cost of implementation and effectiveness in reducing the risk of closure are combined to find the benefit-cost ratios for some mitigation options.

Chapter 9 describes the development of decision analysis software that can be used to determine the spending portfolio that optimises risk reduction benefits for a given expenditure. This software has potential uses in preliminary investigation of mitigation options.

Chapter 10 summarises the findings and conclusions that were drawn from the study.

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2. CLOSURE DUE TO SNOW AND ICE

2.1 Introduction

Both the geographic location and high elevation of the Desert Road mean that it is susceptible to closure in extreme weather conditions due to the presence of snow and ice on the road. Whilst snow is easily dealt with by traditional road maintenance vehicles which can generally clear the road rapidly once the snow ceases to fall, ice is far more difficult to remove. For this reason, the presence of ice is the dominant influence affecting the frequency and duration of road closure during extreme weather (Works Consultancy Services, 1996).

The formation of ice is a complex function of heat flow and moisture conditions at the road surface. Ice forms on roads through four main mechanisms:

- snow melting on the road and then refreezing;
- rain falling on the road and freezing;
- the freezing of moisture saturated air blowing across the road;
- or the compaction of snow by passing vehicles.

The degree to which a section of road is at risk from snow and ice will be determined by the road's characteristics. These include the thermal properties, colour and reflectivity of the pavement, and the prevailing atmospheric conditions including air temperature, humidity, wind, precipitation, evaporation, and solar radiation at the site (Logan, 1992).

Over much of its length the Desert Road is an exposed environment, where the presence of wind and sunlight limit the duration of an icing event. There are however several sections of the road which are continuously both sheltered and shaded by bluffs or vegetation during winter. In such locations ice may not fully melt during the day, often refreezing to a greater thickness upon nightfall (Works Consultancy Services, 1996).

2.2 Modelling Techniques

Before the likely costs of road closure due to snow and ice can be quantified, the susceptibility of the road to closure by this means needs to be assessed. This could be done by either statistically analysing the historical frequency of road closure due to snow and ice conditions; or by identifying the causative mechanisms that induce closure of the road and finding the frequency with which these occur. The problems inherent with both methods are discussed.

2.2.1 Historical Frequency of Road Closure

In instances where there are sufficient data available, the expected frequency and duration of road closure can be established from what has occurred in the past. For this method to be valid two key conditions must be met. There should be a reliable

record of adequate length describing what has occurred in the past; and the conditions under which the data were gathered must have not changed significantly during that period, or be expected to change in the future.

Whilst there are records available detailing closures of the Desert Road, the second requirement for this type of analysis appears not to be met by the data. The changing social and political environment under which Transit New Zealand operates, indirectly dictates the road conditions that are deemed to justify road closure. The level of service and safety expected by the travelling public has also increased. Should road conditions fall below the current danger threshold, Transit New Zealand is legally bound to close the road. Another factor affecting closure times is the implementation of the 1992 Health and Safety in Employment Act, with which Transit New Zealand must abide when contractors are clearing the road. Thus the weather conditions that create dangerous conditions and lead to closure in today's environment may not have led to the same length of closure in previous decades.

2.2.2 Frequency of Causative Mechanisms

An alternative approach to modelling the risk of road closure is to identify the causative mechanisms that initiate closure and assess the frequency with which they occur.

If the weather conditions that lead to road closure under today's closure criteria can be identified, then these weather conditions can be used to evaluate the frequency with which the road would have closed in the past, given the present closure criteria. In many instances the record of weather information will be much longer than the record of road closures, and therefore provide a more substantial database. Thus instead of using historical road closure data, whose criteria become more uncertain with increasing time, historical weather data are used.

2.3 Snow and Ice Model Development

As discussed above, the nature of historical road closure data is not suitable for simple statistical analysis. The risk that snow and ice impose must therefore be quantified by identification and analysis of the causative mechanisms. This was completed in four main stages. Maximum and minimum daily temperatures were identified as the prime indicator parameters, and the temperatures required for icing conditions were defined. In the next stage, a correlation was developed between these icing conditions and the length of the resulting road closure. The frequency with which these icing conditions occur was then found, allowing the frequency of particular road closure durations to be estimated. Further development of the model then incorporated the likelihood of consecutive closure of the alternative routes to the Desert Road.

Although it would have been useful to model snow and ice as two separate closure mechanisms for closure mitigation planning, the pair are very closely related. Many of the stated reasons for historical road closures are a combination of both of these factors. Separation of the closure data effectively limits the quantity of data

available for the analysis of the individual closure mechanisms. Thus the division of these two causative events would greatly increase the levels of uncertainty incorporated in the closure model.

2.3.1 Definition of Road Closure Conditions

The initial stage of model development involved identifying the weather conditions required for the road to be closed due to snow and ice. Unfortunately there is no weather recording station on the Desert Road. Because of this, weather data from the nearby township, Waiouru (see Figure 1.1) were used. The summit of the Desert Road is approximately 300m higher than Waiouru, and thus experiences more extreme temperatures. Consequently, the weather data from Waiouru are not identical to that which would have been experienced at the Desert Road. The same general weather patterns would apply at both areas though, with any severe storm event at the Desert Road also affecting Waiouru. Weather conditions at Waiouru are therefore assumed to be related to those at the Desert Road.

Figure 2-1 shows maximum and minimum daily temperatures plotted against relative humidity at 9am on days of Desert Road closure during 1994, 1995 and 1996 (total of 55 observations). These weather conditions were measured at a recording station in Waiouru. General conditions required for icing are surface temperatures below zero degrees Celsius combined with high humidity (Thornes and Shao, 1991). From Figure 2-1 it can be seen that closure occurred on days when the daily minimum temperature at Waiouru did not reach zero degrees. This is because Waiouru is at a lower elevation than the Desert Road and is thus a few degrees warmer. Raising the minimum temperature defining icing conditions to two degrees Celsius effectively adjusts the Waiouru data for the elevation difference.

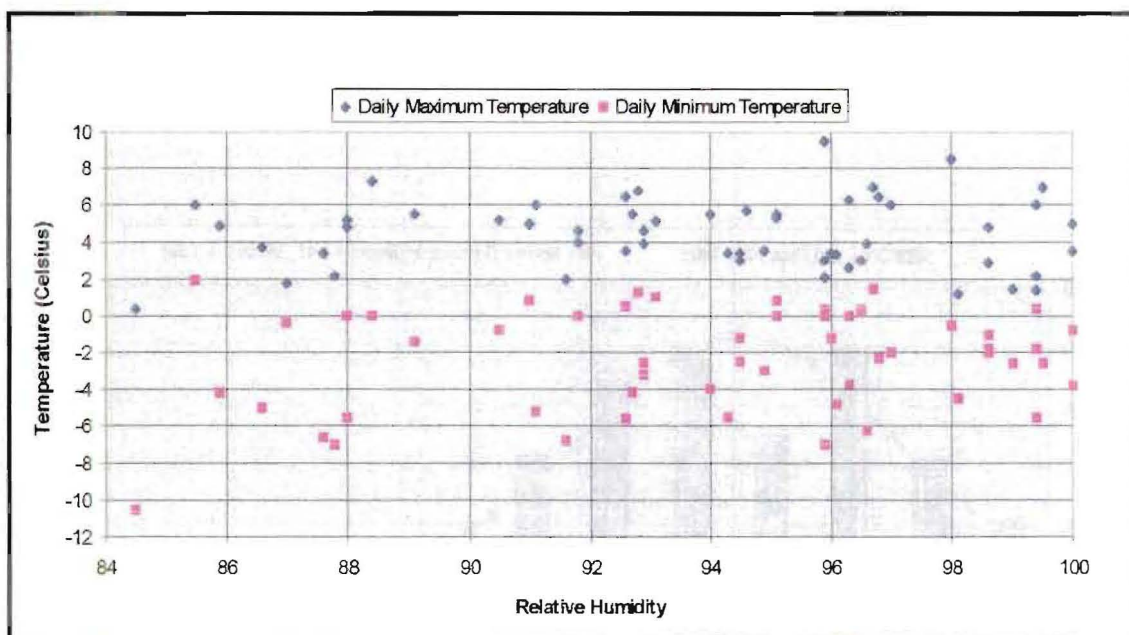


Figure 2-1: Weather conditions on days during which there was road closure (1994-1996)

Closure criteria based on the daily minimum temperature alone is not a reliable indicator of road closure, as a large proportion of days in the weather record fitting this criterion do not coincide with road closure. This generally occurred when the temperature range was large, such as occurs in Autumn and Spring, with clear sunny days preceded by a frost. To eliminate these frost days, ice formation conditions were based on days with a minimum daily temperature falling below two degrees Celsius and a maximum daily temperature below ten degrees Celsius. All days on which road closure occurred met these criteria.

When determining the temperatures that define icing conditions, there is uncertainty whether the temperatures selected are appropriate. It is necessary to determine the degree of confidence there is in the temperatures chosen. To do this a frequency distribution was plotted of both the maximum and minimum temperatures that led to closure in 1994-1996. Statistical tests were used to identify a distribution that adequately characterised these data. It was found that both of the temperature distributions were best represented by the normal distribution, with the parameters shown in Figure 2-2.

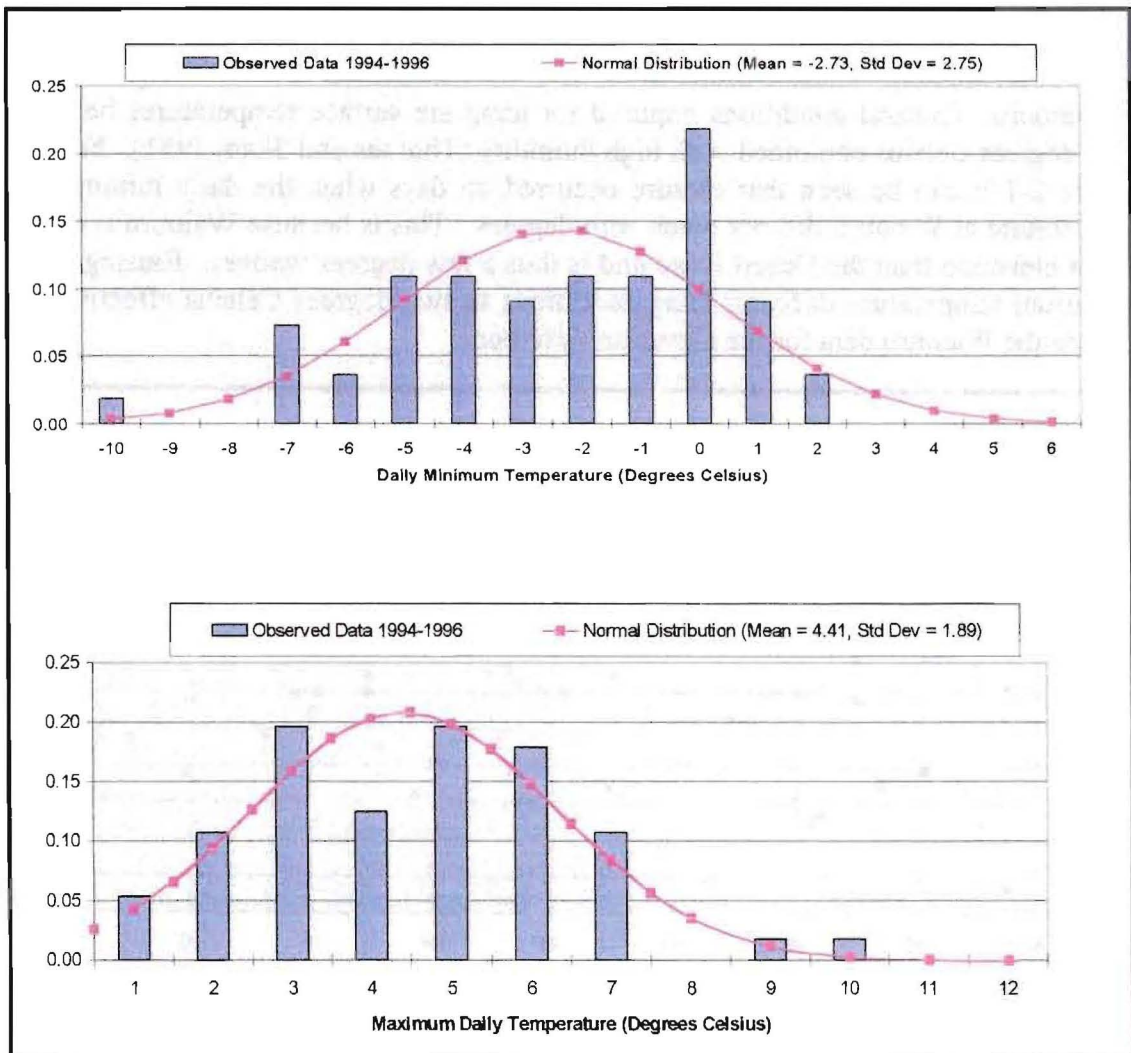


Figure 2-2: Fitting of a Normal Curve to the Daily Maximum and Minimum Observed Temperatures during Closure 1994-1996

The probability that a minimum temperature above 2 degrees, or a maximum temperature above ten degrees would lead to road closure was then determined. It was found that there is 94.6% certainty that the true minimum temperature for icing falls below 2 degrees Celsius, indicating that this criterion is reasonable. The maximum daily temperature criterion was found to be conservative, with 99.85% certainty that the true criterion falls below 10 degrees.

The presence of ice formation conditions does not always result in road closure, with only twenty-five percent of days meeting these criteria resulting in closure. By defining the conditions required for icing, the data to be analysed is limited to only those days that have any significant chance of resulting in road closure.

2.3.2 Icing Conditions and Closure Duration

The length of time that the road will be closed due to snow and ice is related to the severity of the causative mechanisms. In this case the severity or magnitude of an event has been defined as the number of consecutive days conforming to the ice formation criteria. For example, a magnitude one event would consist of only one day of icing conditions, whilst an event of magnitude eight would occur when icing conditions exist for eight days in a row.

A magnitude eight event would have a higher possibility of causing road closure than a magnitude one event, as there are more nights available for ice to form on the road surface. Sustained icing conditions for long periods are also likely to result in longer road closures. This is because warmer temperatures required to melt ice from the pavement surface are not occurring over this period. Therefore a possible relationship exists between the event magnitude and both the probability of road closure and the duration of any closures.

It could be argued that icing conditions followed by sustained low temperature conditions and cloud cover could result in extended closure, yet would not meet the minimum daily temperature closure criteria. This eventuality would account for an increased variance of closure durations associated with a given magnitude event.

Table 2-1 gives observed probabilities of road closure for specified ice durations. This shows that the longer icing conditions are present, the more likely the road will close. This is summarised by the general equation:

$$\begin{aligned} P(\text{closure}) &= \text{fn}(\text{Duration of icing conditions}) \\ P(C) &= \text{fn}(D_I) \end{aligned} \tag{2-1}$$

where $P(C)$ and D_I are defined in Table 2-1.

Table 2-1: Observed probabilities of road closure for each icing duration (1994-1996)

Duration of Icing Conditions, D_I (days)	No of Episodes (of D_I) Meeting Icing Criteria (1994-1996)	No of Episodes (of D_I) that had one or more Road Closures	Proportion of Episodes (of D_I) Meeting Closure Criteria on which there was a Closure, $P(C)$
1	32	0	0
2	11	1	0.09
3	5	3	0.60
4	7	5	0.71
5	6	2	0.33
6-12 (Ave = 9)	6	5	0.83

To find the relationship between the duration of icing conditions and the probability of closure, three approaches are trialled. These are discussed below.

The duration of icing conditions is plotted against the observed probability of road closure, and a linear trendline fitted through the observed data points (Figure 2-3). This allows the relationship between the two parameters to be defined. As the number of observations increases, the observed proportion established by these observations will tend towards the true proportion of road closures that will result from each icing duration. Thus when plotting the linear trendline through the observed proportions, each data point is weighted according to the number of observations used to establish the proportion (i.e. a weighted least squares regression).

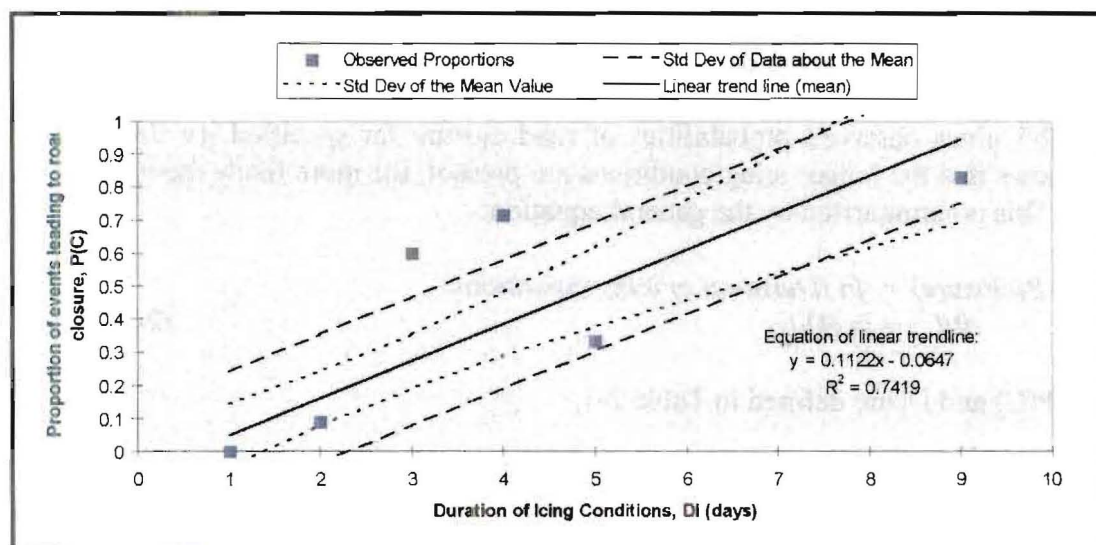


Figure 2-3: Fit of a linear regression trendline to the observed proportions

For each icing duration (D_I), the observed proportion of time that the road will close [$P(C)_{DI}$] will vary normally about the mean proportion [$P(C)_{DI}$] predicted by the linear trendline. The variance of these distributions is found by:

$$s^2 = \frac{1}{n-2} \sum_{D_I=1}^n w_{D_I} \left[P(C)_{D_I} - \overline{P(C)_{D_I}} \right]^2 \quad (2-2)$$

where w_{DI} is the weighting assigned to each data point, and $\overline{P(C)_{DI}}$ is the mean proportion defined by the linear trendline. (Sen and Srivastava, 1990; Benjamin and Cornell, 1970)

There is also uncertainty surrounding the appropriateness of the regression line variables. Estimates of the mean proportion for each icing duration will be better for those icing durations near the mean of the observed range (i.e. the first moment of the observed data points). Icing durations that are outside the range of observations will be more uncertain, as there is no evidence to show that the regression is relevant outside of this range. For this reason, the uncertainty about the regression line will increase with distance from the first moment of the data points, and is calculated as:

$$s_{\overline{P(C)_{D_{Io}}}}^2 = \frac{s^2}{n} \left[1 + \frac{(D_{Io} - \overline{D_I})^2}{s_{D_I}^2} \right] \quad (2-3)$$

where $s_{\overline{P(C)_{D_{Io}}}}^2$ is the variance about the regression line predicting the average proportion of events leading to closure $\overline{P(C)_{DI}}$ for the icing duration D_{Io} , and $\overline{D_I}$ is the first moment (mean) of the observed icing durations. The standard deviation $s_{\overline{P(C)_{D_{Io}}}}$ is shown as the dashed curve in Figure 2-3. For a given value of D_I then, the variation of an individual value $P(C)_{DI}$ is the sum of these two uncertainties (Equation 2-2 and 2-3) (Benjamin and Cornell, 1970).

A limitation of the above linear regression model is that it is possible for the model to generate the probability of an event leading to closure as greater than one, or less than zero. In these situations where the normal distribution indicates that there is a possibility of negative probability, the probability is taken to be zero (see Figure 2-4). Similarly, probabilities greater than 1.0 are assumed to equal one.

The reader should note that although this linear regression model predicts that on average, icing conditions for 10 days or more will always result in closure, variance about this average proportion means that the model will predict some times where these events do not result in closure. The chance of this though will decrease as the number of icing days increases.

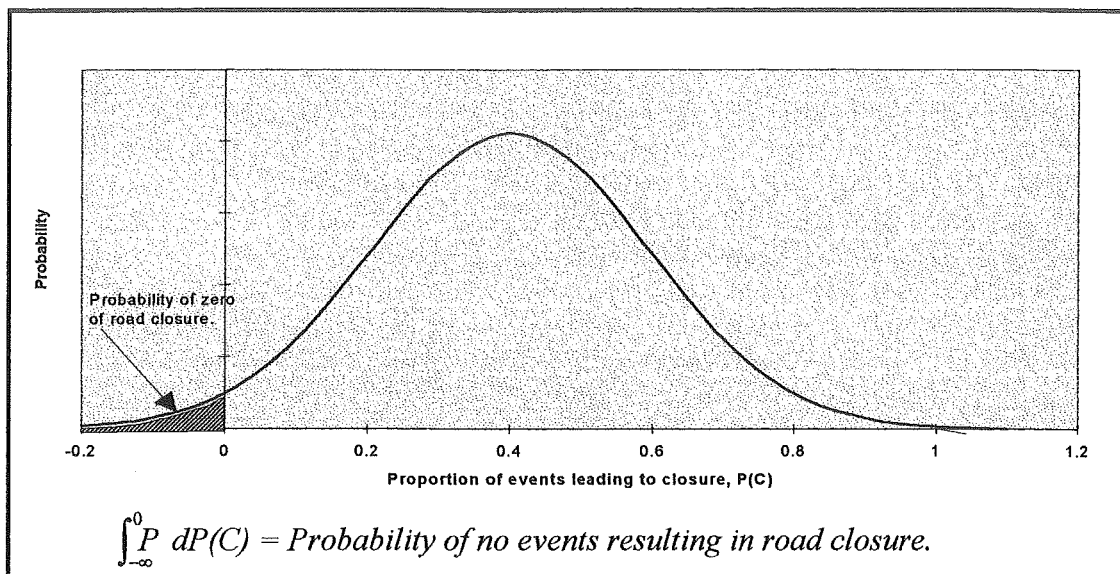


Figure 2-4: Illustration of the "pooling" of negative closure durations at zero

Although the regression line (Figure 2-3) does not fit the observed data as well as would be desired, this lack of fit will be reflected in the predictions of the model. The better the fit of a regression line, the less variance there will be in predictions that are generated using the regression. Where a regression may not be ideal, the lack of fit of the regression line will be reflected in the model outputs, with a large spread in the range of model predictions. This means that no accuracy greater than is actually present is implied by the model results.

A possible alternative way to model the probability of an event leading to closure would have been to use Bernoulli trials. Bernoulli trials have the advantage of always predicting a probability of closure between 0 and 1. The probability of an icing event of duration D_I , resulting in road closure would then be:

$$P(C) = 1 - (1 - P(C_o))^{D_I} \quad (2-4)$$

where D_I is the duration of icing conditions, and $P(C_o)$ is the probability that any day with icing conditions will result in road closure, irrespective of the total duration of icing conditions. Although this model would eliminate the prediction of probabilities outside the bounds of 0 and 1, it does not fit the observed data as well as the linear regression model. This may indicate that the likelihood of a day with icing conditions resulting in road closure is not independent of the duration of icing conditions. The Bernoulli trial approach was thus not preferred above the linear regression model.

The third approach is that the probability of an event leading to closure is related to the duration of icing in a negative exponential type manner. As the duration of icing conditions increases, the chances of closure increase asymptotically to one. This relationship is described in Equation 2-5.

$$P(C) = 1 - \exp(\text{constant} \times D_I) \quad (2-5)$$

The appropriate value for the constant in Equation 2-5 is that which optimises the R^2 value for the fit of this relationship to the observed data. When the constant is set to be -0.14, an optimum R^2 value of 0.72 is attained. Figure 2-5 shows the relationship plotted against the observed data.

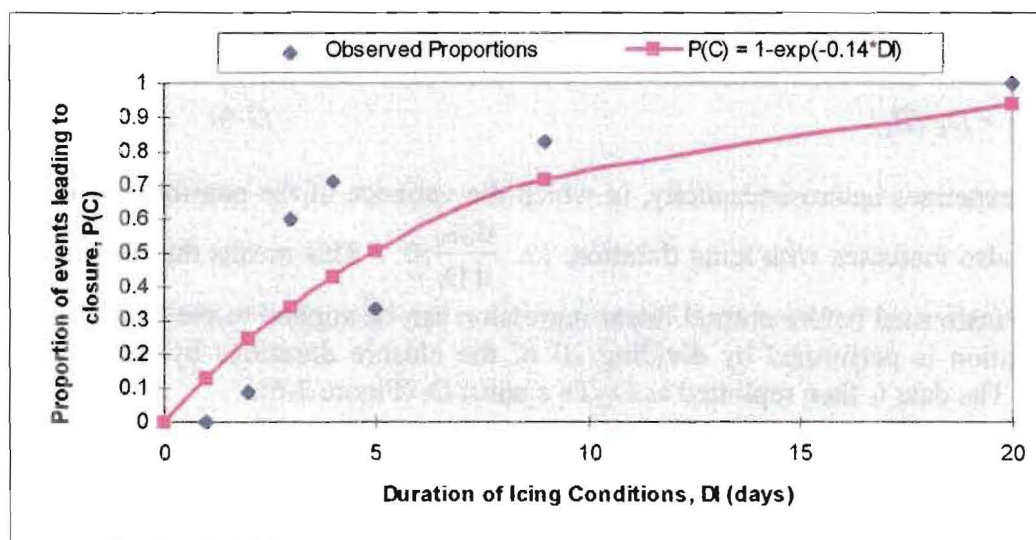


Figure 2-5: Fit of the negative exponential relationship to the observed data.

There are both advantages and disadvantages of modelling the proportion of events leading to closure as a negative exponential relationship. The most significant advantage is that the proportions predicted are bound between 0 and 1. A disadvantage of this relationship though is that the curve passes through the origin and will therefore always overpredict the proportion of single day icing events that result in closure. In this case the relationship predicts that 13% of single day icing events will result in closure. Given that out of an observed sample of 32 events, not a single event resulted in any closure, this prediction seems much too high.

Both the linear regression model and the negative exponential relationship have their disadvantages. It is therefore necessary to assess the impact of each of these drawbacks on the final model predictions. The linear regression model predicts that for events with an icing duration greater than 10 days, the proportion of events leading to closure is greater than one (this is then assumed to be equal to 1.0). The negative exponential relationship is fine for the longer duration events, but predicts significantly higher proportions of short duration events resulting in closure. Given that shorter duration icing events happen quite frequently, errors in the prediction of their propensity to lead to closure will be more significant than the smaller errors for longer duration events. This indicates that the linear regression model is more suitable.

The linear regression model is certainly not the ideal model for predicting the proportion of events leading to closure, not least because it is not bounded between 0 and 1. Other models that have been tried however also have significant limitations, and they do not represent the observed data as well as this linear regression model. For this reason it was decided to continue modelling with the linear regression

model. If further refinement of the model predictions is required in the future, this is an area that would benefit from further work.

If an icing event does result in road closure, the question then remains as to how long the road will be closed. Resulting closure duration (D_C) will be a function of the time over which icing conditions exist (D_I), with closure duration increasing with icing duration.

$$D_C = D_C(D_I) \quad (2-6)$$

The data expresses heteroskedasticity, in which the variance of the possible closure durations also increases with icing duration: i.e. $\frac{d\sigma_{DC}}{dD_I} > 0$. This means that the data must be transformed before normal linear regression can be applied to the data. The transformation is performed by dividing all of the closure durations by the icing duration. The data is then replotted as D_C/D_I against D_I (Figure 2-6).

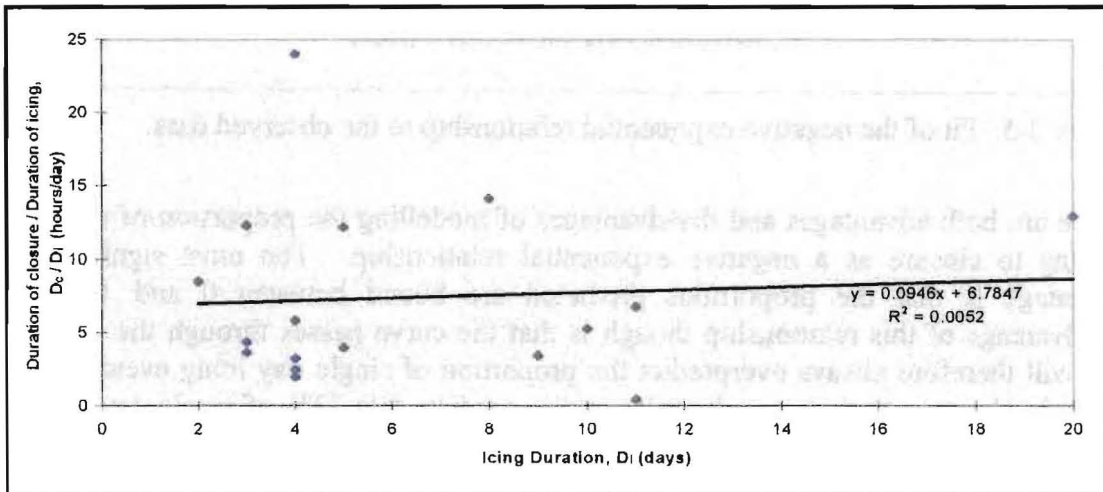


Figure 2-6: Plot of the transformed closure durations against icing duration. The nearly horizontal trendline indicates approximate independence

Note D_C/D_I has units of hours per day, since road closures are expressed in hours and icing in days. The standard deviation of this new data set is homoskedastic, permitting linear regression to be used. The regression indicates that D_C/D_I is approximately independent of D_I ($R^2=0.0052$), allowing the probability distribution of D_C/D_I to be defined for all icing durations.

The Kolmogorov-Smirnov statistical test was used to establish the best empirical distribution to fit the observed D_C/D_I data. Only those distributions that reflected the physical bounds that D_C/D_I must be between zero and twenty-four hours of closure in any one day were tested. It was found that the Beta distribution (Equation 2-7) with the parameters $\alpha_1=0.74$, and $\alpha_2=1.67$, multiplied by the scaling factor 24, satisfied the Kolmogorov-Smirnov test at a 90% level of significance. The shape of this distribution is illustrated in Figure 2-7.

$$f\left(\frac{D_C}{D_I}\right) = \frac{\frac{D_C}{D_I}^{\alpha_1-1} \left(1 - \frac{D_C}{D_I}\right)^{\alpha_2-1}}{B(\alpha_1, \alpha_2)} \quad (2-7)$$

$$B(\alpha_1, \alpha_2) = \int_0^1 t^{\alpha_1-1} (1-t)^{\alpha_2-1} dt$$

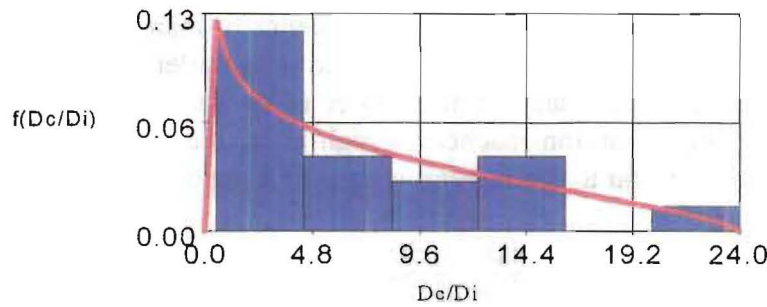


Figure 2-7: Comparison of the observed data and the Beta distribution (0.74,1.67)*24

Possible Alternative Modelling Approach

In the approach that has been described above, closure of the Desert Road has been predicted using weather conditions at Waiouru. An alternative approach would be to use freezing level data to predict closure of the road. The freezing level is determined by air temperatures at different elevations in the atmosphere, recorded using weather balloons. This data has been recorded at Paraparaumu, near Wellington. This location is within one degree latitude of the Desert Road, and should be representative of the atmospheric temperatures over the region.

Freezing level data will provide information on general weather conditions over a large area. In contrast, temperatures at Waiouru provide information on weather conditions in the localised Central Plateau area. Both sets of information will provide some indication as to the propensity of the Desert Road to be affected by snow and ice. The question is which data set is more appropriate to this analysis. To determine this, modelling was carried out using the freezing level data and the results were then compared to the outputs from the Waiouru temperature model. Unfortunately, the record of freezing levels at Paraparaumu is incomplete for 1996, so the analysis used only two years worth of data, 1994 and 1995, to examine the relationship between weather conditions and road closure.

The freezing level that was observed at Paraparaumu on days of closure of the Desert Road were analysed to find the defining freezing level that is required for the Desert Road to close. The 95th percentile of these observed freezing levels was taken to be the condition required for icing on the road. This threshold, used as the icing criteria, was a freezing level below 1650m. The freezing level record was then

checked to see how many days meeting this condition actually resulted in road closure. It was found that only 16% of days meeting these criteria resulted in road closure. This compares to 25% of the days meeting the Waiouru temperature criteria resulting in closure, meaning that the freezing level icing criteria is less defining than the Waiouru temperature criteria are.

Using the freezing level criteria, the historical record was then assessed to see if there is a relationship between the duration of freezing conditions and the proportion of events leading to road closure. Linear regression was used to find this relationship, with an R^2 value of 0.87. This was a better fit than that obtained using the Waiouru data. In part, this could be attributed to the smaller sample size and the greater spread in the observed data. The number of consecutive days of freezing levels meeting the icing condition reached as high as 17 days, whilst the Waiouru regression only spanned as far as 8 consecutive days of icing conditions.

The duration of road closures were then analysed to see if they relate to the freezing levels observed at Paraparaumu. The duration of any road closure that resulted appeared to have little relationship to the duration of consecutive days meeting the freezing level icing criteria. There were times where the freezing level criteria were met for as many as 14 days in a row, yet the road was closed for only half a day, and yet others where the freezing conditions existed for shorter periods, but resulted in longer road closures.

In all, there is little evidence to suggest that a better model will be achieved by using freezing level data to predict closures of the Desert Road by snow and ice conditions. For this reason, analysis using the Waiouru data was continued.

Another alternative would be to use daily weather data recorded at the Chateau (see Figure 1.1), on the other side of Mount Ruapehu from the Desert Road. The Chateau is at a similar elevation to the Desert Road, though may experience different weather conditions due to the influence of the mountain. The weather record from the Chateau does not span past 1992 however, making this approach unsuitable.

2.3.3 Probability of Icing Conditions

Weather data from Waiouru for the twenty-one years prior to 1993 were used to assess the typical weather patterns of the area. In order to project the frequency of ice formation conditions from this data forward into the next twenty-five years, the weather patterns must be regarded as statistically stationary. There are however long term weather fluctuations.

The Greenhouse effect is leading to global warming to the extent of approximately 0.1-0.25 degrees Celsius per decade (Houghton, 1994). In New Zealand, mean annual temperatures have been rising on average by 0.11 degrees Celsius per decade (Zheng *et al.*, 1997). The associated change in weather patterns due to global heating are difficult to predict, but it is recognised at temperate latitudes they may result in extreme weather patterns. The temperatures measured during historical road closures (Figure 2-1) indicate that this temperature elevation is unlikely to be statistically significant. Over the 25 year analysis period, the expected increase in

mean temperature of 0.275 degrees Celsius is small compared to the range of temperatures observed on days of closure.

Oscillation between the El Niño and La Niña effects will introduce a periodicity to winter temperatures. These processes are associated with periodic fluctuations in atmospheric pressures, affecting weather patterns over a 3.8 year cycle (Philander, 1990). The temperature change is a result of the changing weather patterns which in turn are affected by the global heat budget. The temperature record used in the analysis of closure of the Desert Road was of 25 year duration, which is much longer than this cycle. This means that at least 8 weather cycles due to El Niño and La Niña occurred during the period of the weather record. Consequently variability due to this source was incorporated in the model predictions.

For the purposes of the model developed here, the historical weather data shall be taken as representative of future weather patterns. Available weather records from Waiouru span back to 1972. The full length of this record up until 1993 was used for this analysis, except for 1983, when the data set was incomplete.

A lognormal distribution was used to model the frequency of icing conditions per year (F), since F must always have a positive value. For each of the icing durations (D_i), both the mean and the standard deviation of the log of the frequencies $[\ln(F)]$ were computed (Figure 2-8). A linear trendline, plotted through the mean of the log of the frequencies $[\ln(F)]$, had a correlation coefficient of 0.97. This linear relationship was used to define a lognormal distribution for each of the icing durations. There was little dependence between the standard deviation of the logged frequencies and icing duration, indicating the homoskedasticity of the data. The standard deviation was taken to be the average of the observed standard deviations.

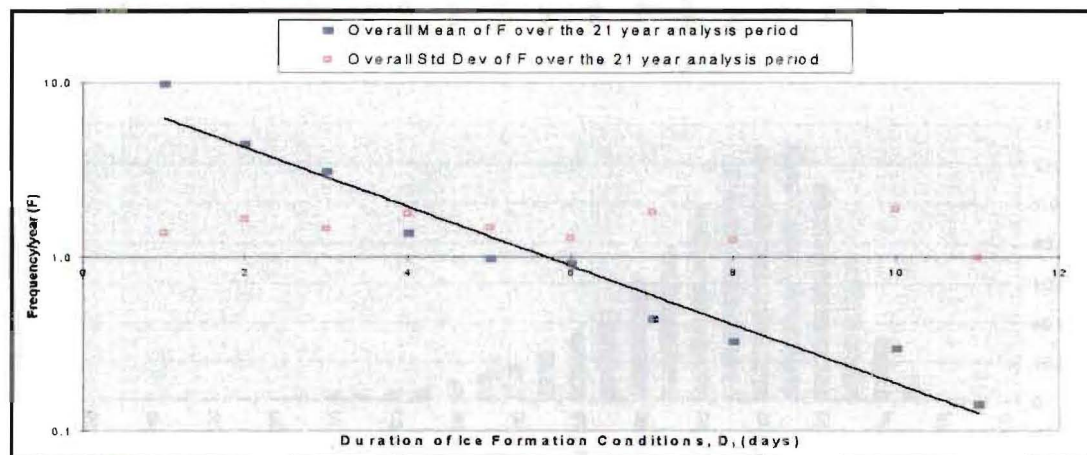


Figure 2-8: The mean and standard deviation of the logged yearly frequency of each icing duration, over the 21 year analysis period

The relationship between icing duration and its frequency of occurrence to be used in the risk or probability model is then:

$$f(F_{D_I}) = \frac{1}{F_{D_I} \sigma_{\ln(F)} \sqrt{2\pi}} \exp \left[-\frac{1}{2} \left(\frac{\ln(F_{D_I}) - \mu_{\ln(F_{D_I})}}{\sigma_{\ln(F)}} \right)^2 \right] \quad (2-8)$$

where $\mu_{\ln(F_{D_I})} = -0.3903 D_I + 2.229$, and $\sigma_{\ln(F)} = 0.4$, and F_{D_I} is the frequency of occurrence of icing duration D_I .

2.3.4 Combining the Frequency and Closure Duration Relationships

In the previous sections it was described how for each event magnitude (icing duration), both the frequency of occurrence of that icing duration, and the road closure durations expected to result were estimated. The first graph in Figure 2-9 shows the probability distribution of closure durations likely to result from a magnitude 8 event, generated using Equation 2.7. The second graph shows the probability distribution for the frequency with which a magnitude 8 icing event would occur. This was generated using Equation 2.8.

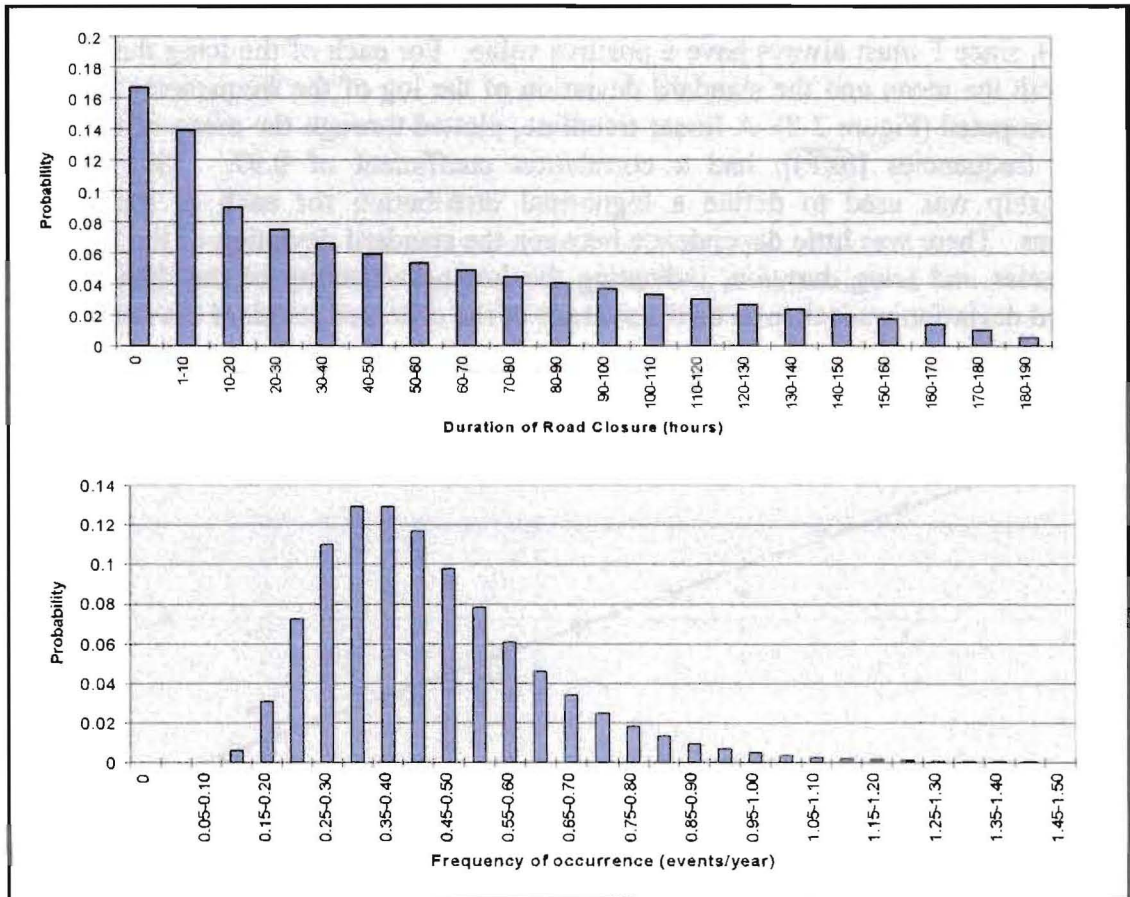


Figure 2-9: The frequency of occurrence and duration of closure distributions for a magnitude 8 event (8 days of consecutive ice formation conditions)

Using Monte-Carlo simulation (randomly generating possible scenarios arising from the two relationships) the probability of frequency of occurrence and closure duration combinations can be found. The computer program @RISK (Palisade

Corporation) was employed for this purpose. There are two methods that may be used by @RISK to perform this, depending upon the type of distribution being modelled. If the distribution is well defined and invertible, then the transformation method is used (Figure 2-10). To do this the computer generates a random number between 0 and 1, which is then compared to the cumulative frequency distribution (cdf) of the parameter being modelled. The value of the parameter that aligns with this randomly generated cumulative frequency is selected. The steep slope of the cumulative frequency distribution about the mean value leads to these values being selected more regularly.

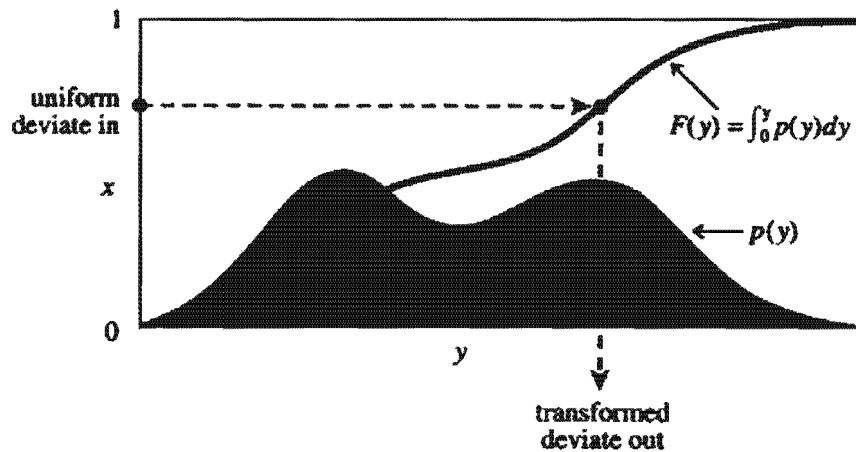


Figure 2-10: Transformation method for generating a random deviate y from a known probability distribution $p(y)$ (Press *et al.*, 1992)

Alternatively, if the cdf is complex and difficult to invert, the rejection method of parameter selection is used (Figure 2-11). This utilises the probability density function (pdf) of the parameter. Initially, a comparison function is established, which lies everywhere above the original pdf. The cdf of this comparison function is then determined. A random number is generated, between 0 and A , where A is the area beneath the comparison distribution. The random number is aligned with the cdf of the comparison function, to select a parameter value. Another random variable is then generated between 0 and $f(x)$, where $f(x)$ is the value of the comparison function at the selected parameter value. If the second generated value is less than the value of the actual pdf at that point, then the parameter is selected to be used in the model. If the second parameter does not fall within the bound of the original pdf, then that parameter is rejected (Press *et al.*, 1992).

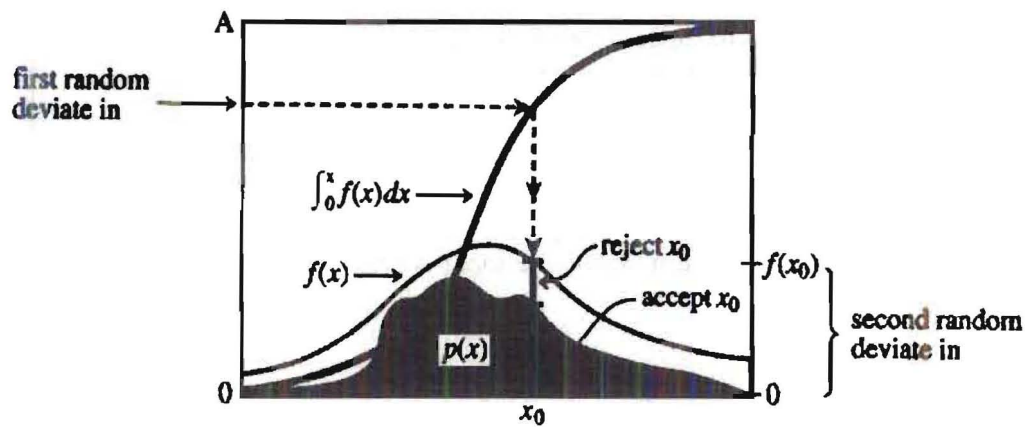


Figure 2-11: Rejection method for generating a random deviate x from a known probability distribution $p(x)$ (Press *et al.*, 1992)

With either method, the process is repeated many times. Figure 2-12 shows some possible scenarios for the frequency of occurrence of different closure durations due to a magnitude 8 event predicted by the model.

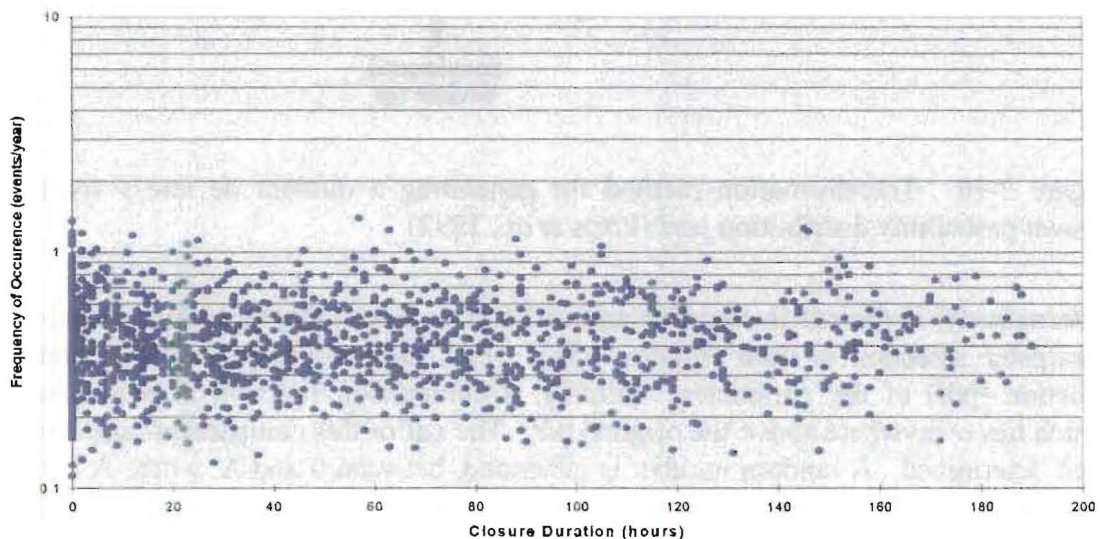


Figure 2-12: Relationship showing the number of occurrences per year of road closure durations due to a magnitude 8 event

As the occurrence of separate magnitude events are assumed to be independent, the total cost of closures due to snow and ice can be calculated as the sum of the costs incurred by each of the event magnitudes. The average closure duration and frequency of occurrence of a range of event magnitudes are plotted together in Figure 2-13 to illustrate the comparative risks imposed by each of the event magnitudes.

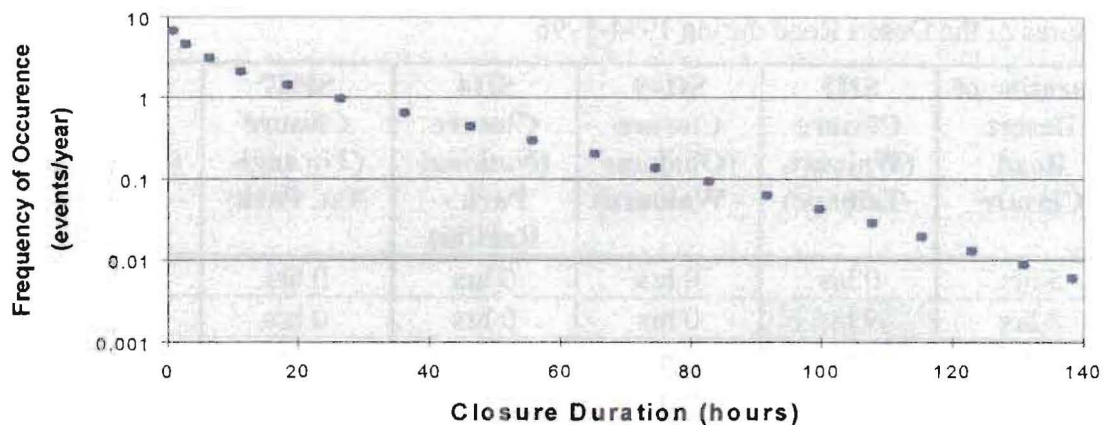


Figure 2-13: Average closure durations plotted against the average frequency of occurrence for all event magnitudes ranging from one day to twenty days of icing conditions

2.3.5 Status of Alternative Routes

When the Desert Road is closed it is possible that the alternative route a traveller may wish to use will also be closed. This then forces the traveller to take an even longer path to their destination, increasing the cost to society of the Desert Road closure. It is therefore necessary to find the probability and consequences of the alternative routes closing at the same time as the Desert Road.

The roads that are likely to close simultaneously with the Desert Road are State Highway (SH) 1 to the south of the Desert Road between Taihape and Waiouru, SH 49, SH 4, SH 47 and SH 41 between Manunui and Turangi (see Figure 1.1 for SH locations).

One approach would be to relate the weather conditions at Waiouru to closures of the alternative routes. The increased spatial difference and smaller data set available for this comparison however result in large uncertainties in the relationships between the two. An alternative approach is to relate road closures on the Desert Road with the probability and consequences of closure on the alternative routes. This is a less mechanistic approach, but has the advantage of reducing the numbers of parameters involved, and thereby the total error in the estimates.

As the objective is to find the costs of Desert Road closure, only those closures on the alternative routes that coincided with Desert Road closure were incorporated. Closures from the period 1994 to 1996 were used in the analysis.

The closure data (Table 2-2) indicates that there are differences between the likelihood of closure of each of the individual roads. State Highway 1, from Taihape to Waiouru is more likely to be concurrently closed with the Desert Road than the more northern highways. This may be a function of the typical weather patterns that cause icing conditions on the Desert Road. The more western and northern routes may be partially sheltered from this weather by the mountains.

Table 2-2: Duration of concurrent road closures on the alternative routes during closures of the Desert Road during 1994-1996

Duration of Desert Road Closure	SH1 Closure (Waiouru - Taihape)	SH49 Closure (Ohakune - Waiouru)	SH4 Closure (National Park - Raetihi)	SH47 Closure (Turangi - Nat. Park)	SH41 Closure (Manunui - Turangi)
5 hrs	0 hrs	0 hrs	0 hrs	0 hrs	0 hrs
8 hrs	9 hrs	0 hrs	0 hrs	0 hrs	0 hrs
10 hrs	0 hrs	0 hrs	0 hrs	0 hrs	0 hrs
11 hrs	0 hrs	0 hrs	0 hrs	0 hrs	0 hrs
13 hrs	0 hrs	0 hrs	0 hrs	0 hrs	0 hrs
17 hrs	0 hrs	0 hrs	0 hrs	0 hrs	0 hrs
20 hrs	14.5 hrs	0 hrs	0 hrs	0 hrs	0 hrs
23.5 hrs	0 hrs	0 hrs	0 hrs	0 hrs	0 hrs
31 hrs	0 hrs	0 hrs	0 hrs	0 hrs	0 hrs
37 hrs	0 hrs	0 hrs	0 hrs	0 hrs	0 hrs
53 hrs	0 hrs	0 hrs	0 hrs	0 hrs	0 hrs
61 hrs	0 hrs	0 hrs	0 hrs	0 hrs	0 hrs
75 hrs	30.5 hrs	10 hrs	0 hrs	0 hrs	0 hrs
96 hrs	0 hrs	0 hrs	0 hrs	0 hrs	0 hrs
113 hrs	6 hrs	8 hrs	8 hrs	0 hrs	0 hrs
259 hrs	27 hrs	27 hrs	20 hrs	27 hrs	27 hrs

The number of alternative roads that are concurrently closed tends to increase with duration of the Desert Road closure. To define this relationship, the Desert Road closure duration was plotted against the number of alternative routes concurrently closed (Figure 2-14).

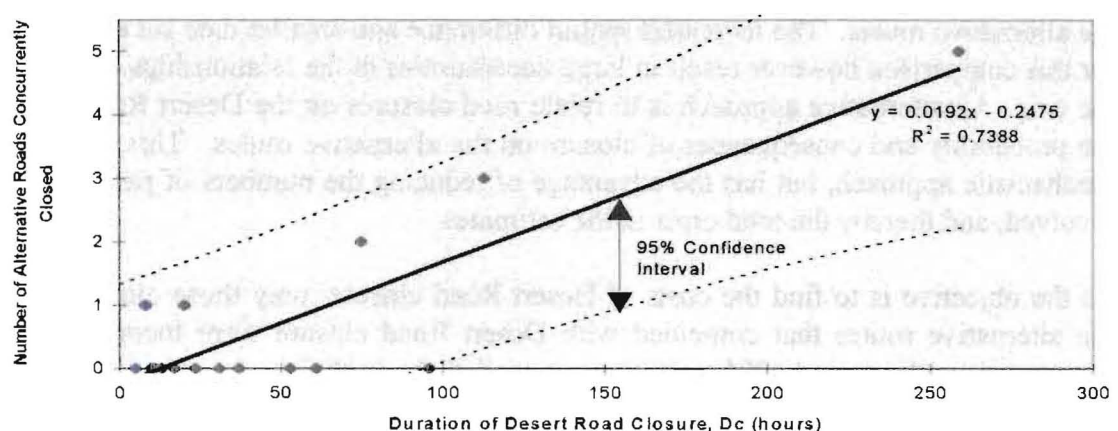


Figure 2-14: Relationship between the number of alternative routes concurrently closed and the duration of Desert Road closure

A trendline was fitted through these points to establish the “mean” number of alternative routes (rounded to the nearest whole number) closed for a given duration

of Desert Road closure. For each Desert Road closure duration (D_c), the observed number of alternative roads concurrently closed will vary normally about the mean number predicted by the linear trendline. The magnitude of this variation was found using Equations 2-2 and 2-3, and is:

$$S_{Total}^2 = 0.4844 + 0.0285 \left[\frac{(D_c - 49.7)^2}{3732} \right] \quad (2-9)$$

The alternative routes that close will follow the order of closure propensity indicated by the data (i.e., SH1 followed by SH49, then SH4, SH47 and SH41).

Although the probability of an alternative road closing increases with the duration of Desert Road closure, there is little relationship between the duration of closure on the Desert Road and the duration of closure on the alternative route. There is also little difference in closure durations between the alternative routes. The independence of closure duration from these two variables allows the probability density distribution of closure times on alternative routes to be established using all of the closure data for the alternative routes from 1994-1996 (i.e., the shaded durations in Table 2-2). The Kolmogorov-Smirnov test for “goodness-of-fit” indicated that the closure duration data is adequately represented by the lognormal distribution:

$$f(D_A) = \frac{1}{D_A \sqrt{2\pi\sigma^2}} \exp\left(-\frac{(\ln D_A - \mu)^2}{2\sigma^2}\right) \quad (2-10)$$

where D_A is the duration the alternative route is closed (in hours); $\mu = 2.73$; $\sigma = 0.58$.

During some of the very long Desert Road closures, the alternative routes were closed intermittently, closing, opening and then re-closing again. As the limited data set inhibits the model from predicting both the number and duration of the closures during a single Desert Road closure, the total closure duration was used. If several short closures were to occur, the costs incurred due to the closures will alter, as waiting times for traffic wanting to use the alternative route is shortened. Conversely, the costs incurred in the actual processes of opening and re-closing the roads will increase. The exact relationship may also depend upon the ability of the travelling public to respond and utilise the open periods, especially if these are short. This may be an area that would benefit from further investigation.

Another assumption that will be used in the costing model, involves instances when multiple alternative routes are closed. When the Desert Road is closed, two or more alternative routes may also be closed. This will limit travellers' options. It shall be assumed that closures of the alternative routes occur concurrently during the Desert Road closure. The 1994-1996 data showed that this is generally the case, however this assumption may slightly exaggerate the projected costs.

2.4 Conclusions

The stochastic model developed characterises the propensity for the Desert Road to close in winter months due to snow and ice on the road surface. It assesses the frequency of occurrence of storm conditions, and determines the distribution of possible road closure durations that result. The effect of storms on the alternative routes are also analysed. Simultaneous closures of these routes will alter drivers' route selection, and thus the costs of any closures.

The model results are useful in the sense that they provide initial estimates of closures of the Desert Road in storm conditions, whilst also explicitly expressing the uncertainties associated with these predictions. If more refined estimates of closures due to snow and ice are required, one area that could be of significant benefit would be to install a weather recording station on the Desert Road. This would remove the necessity for relating icing conditions on the Desert Road to weather conditions in Waiouru.

2.5 References

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3. CLOSURE DUE TO VOLCANIC ACTIVITY

3.1 Introduction

Experience in Washington State following the 1980 eruption of Mount St Helens, clearly illustrated the detrimental effects volcanic activity has upon transportation networks. Immediately preceding the eruption at Mt St Helens, the northern section of the cone collapsed, forming a debris avalanche that covered the northern slopes of the mountain. This debris avalanche extended 24 km down an adjacent valley, permanently burying 32 km of state highway.

The eruption produced a directed lateral blast to the north consisting of shock waves, extremely high temperatures, and massive amounts of airborne pyroclastic debris. The destruction caused by this lateral blast was both sudden and total. There was complete devastation within 13 km of the crater in the direction of the blast. Extending beyond this zone, older trees and most of the smaller new growth were blown down or buried by airborne deposits within 20-24 km of the crater. Within both of these regions all civil works and operations were either destroyed or badly damaged.

Mud and associated debris flowed down several of the valleys radiating from the mountain as a direct result of the May 18 eruption. Several lengths of State Highway were buried up to depths of 2 meters by these mud-flows and many bridges were lost.

Ashfall immediately paralysed transportation in the area. Within hours of the May 18 eruption, 2900 km of State Highways were closed in Eastern Washington due to ash accumulation. A major interstate was closed for a week, and thousands of kilometres of country roads and municipal streets were closed; some for only a few hours, but others for weeks (Schuster, 1981).

There is the potential for all of these hazards to similarly affect transportation systems on the Central Plateau. The consequences of a volcanic event upon the integrity of the road network therefore needs to be assessed.

3.2 Modelling Techniques

The Central Plateau road network has the potential to be affected by several volcanic centres, including Mount Ruapehu, Ngauruhoe, Tongariro, and Taupo (see Figure 1.1). The type of threat posed by each of these volcanos is dependent upon both the characteristics of the volcano, and the location of surrounding roads.

Each volcano is distinct in the sense that their eruptions will differ in style, size, and frequency. This requires that each volcano be modelled separately according to the type of potential threats that they pose. It is not possible in the time frame available

to model the risk from each type of volcanic threat accurately. Efforts have therefore been concentrated upon those which pose the greatest risk.

The main factors that determine the effects of an individual eruption are the volume and rate at which material is ejected. This in turn depends partly upon the chemical composition of the magma. Magma is molten rock that contains gases in solution. The more viscous the magma, the harder it becomes for volcanic gases trapped within the magma to escape. These gases trapped within the magma increase the explosivity of an eruption (Latter, 1985).

There are two principal types of erupted material: lava and pyroclasts. Lava is magma from which most of the gas has escaped, it flows relatively quickly. Pyroclastic material on the other hand is magma that has been blown apart explosively, forming a variety of debris ranging from large blocks and bombs to fine grained ash and dust.

Pyroclastic material may be blasted high into the air during an eruption, to be carried by the wind and finally dropped as tephra. Alternatively it may be blown sideways. This lateral movement is often caused by gravitational collapse of the eruption column, resulting in ground hugging pyroclastic flows and surges, containing hot fragmented material and gases (Moore and Sisson, 1981).

Lahars are rapidly flowing mixtures of rock debris and water (Otway *et al.*, 1995). They are an important secondary product of an eruption. Lahars may be generated by rainfall on unconsolidated tephra slopes, the melting of snow by hot pyroclastic material, or the direct ejection of water from crater lakes.

Toxic gases may also be released during an eruption sequence, but regions of risk are confined. This volcanic by-product is thus unlikely to be of importance for this study.

Only those threats with the potential to affect services on the Desert Road have been investigated. At Ruapehu, this includes modelling the effects of ashfall and lahars, which both have the potential to affect services on the Desert Road. Other potential hazards at Ruapehu are the possibility of sector collapse, or failure of the crater lake wall. Both of these events would result in the formation of devastating lahars, and are thus incorporated into the lahar model. Although a common form of eruption at Ruapehu involves the generation of lava flows, it is unlikely that these would extend far enough to affect any transportation links. A directed blast occurring at Ruapehu has the potential to cause great damage. This possibility shall not be modelled however as there is no direct evidence of a directed blast occurring at Ruapehu in the last 100,000 yrs (Latter *et al.*, 1981). Pyroclastic flows (or glowing avalanches) may have the potential to reach the Desert Road, flowing down confined river valleys. In general, an extremely large eruption would be required to generate such pyroclastic flows, in which case the road is likely to already be closed due to other hazards. This limits the impact and thus the importance of pyroclastic flows on road closures.

Ashfall from an eruption at Mt Ngauruhoe may interrupt services along the Desert Road. This volcano only has the potential to form small lahars, which are unlikely to

reach any of the main roads. Pyroclastic avalanches are one of the greatest hazards that Ngauruhoe poses to human life. These are confined to within 5 km of the crater though, and are not a direct threat to the Desert Road.

Ash from an eruption at Tongariro may also affect services along the Desert Road, and thus requires investigation. The crater lake at Tongariro, Blue Lake, contains 1-1½ million cubic meters of water, all of which could be ejected (Latter *et al.*, 1981). The probability of any lahars formed being large enough to close the Desert Road however is small compared to the risks posed by Ruapehu.

A large scale eruption at Taupo would most certainly affect the Desert Road. It would also however affect a major portion of the central North Island. It is not within the scale of this model to incorporate the effects of such a massive destruction zone. Smaller scale eruptions at Taupo could be modelled for, but the probability of these sized events at Taupo is relatively small compared to the risks posed by the other volcanos in the area.

Indicators of impending volcanic activity include earthquakes, bulging of the flanks of the volcano, and detection of chemical and temperature changes. As was the case at Mount St Helens, nearby areas may be closed to the public if it is thought that these signal the start of a large eruption sequence. During periods of volcanic activity, particularly following an initial eruption, there may be uncertainty as to whether that is all the activity that may be expected. The volcano may simple be warming up for its “big one”. It is this sort of uncertainty that may require a road to be closed as a precautionary measure.

It is often difficult to categorise the reasons for historical road closures during volcanic events into closures due to ash, closures due to lahars, and precautionary closures. For this reason, rather than attempt to model precautionary closures implicitly, their effect on likely closure times have been incorporated into the expected closure times for the actual volcanic hazards.

3.3 Volcanic Ash Model Development

The type of eruption to occur at a volcano will be primarily dependent upon the type of magma that is present, and its fluidity. The less viscous the magma, the easier it is for volcanic gases trapped within the magma to escape. The greatest explosive risks occur when the magma is relatively viscous, as the trapped gases increase the explosivity of an eruption. The viscosity of a magma increases with increasing Silica content of the magma. The main types of magma, ranging from a low to high Silica content, are basalt, andesite, dacite and rhyolite. Most of the magma from Ruapehu, Tongariro, and Ngauruhoe are andesitic, whilst the Taupo magma is rhyolitic.

Many eruptions at Ruapehu, Tongariro and Ngauruhoe are Vulcanian type eruptions, which are characterised by an initial violent phase, that may result in the formation of a tall, ash-filled cloud. The main phase of the eruption is characterised by the ejection of viscous and gas-rich magma, bombs, lapilli, and copious amounts of ash. Plinian type eruptions, which are essentially more explosive and powerful than

Vulcanian eruptions, are also possible at these volcanos. At times when there is crater lake water present, Phreatomagmatic eruptions may occur at Mt Ruapehu and Mt Tongariro. Phreatomagmatic eruptions are violent and explosive eruptions caused by water coming into contact with magma. The high viscosity of Taupo magma means that very explosive Plinian and Ultra-Plinian eruptions are possible.

3.3.1 Probability of Volcanic Activity

A problem with calculating return periods of volcanic activity from evidence of previous eruptions is that often data is either non existent or incomplete. For example, the prehistoric record (i.e. that inferred from geological deposits) is unlikely to record events below a certain sized magnitude due to the relatively small size of deposits. Of the ash that does remain, the integrity of the record to indicate accurately the size of eruptive events is questionable. Erosion of deposit layers is common, creating uncertainties as to the actual volume of material ejected. Eruptive episodes may also occur within a closely spaced interval in geological time, with new deposits being laid over the top of previously erupted material. This total deposit may then be mistaken to be from one larger eruptive sequence. Thus the general quality of the prehistoric record is in doubt, with only evidence preserved from the largest eruptive events viewed with any confidence.

The historic record (i.e. since European settlement) provides data on smaller eruptive events that have occurred during this time. Unfortunately, the inferred volumes of ejected material from these eruptions are also relatively uncertain, with many calculations derived from sketchily recorded human observations. Also the short duration of the record, in relation to the long return periods of low probability events, means that this record is unsuitable for predicting the frequency of larger eruptions.

The historic and prehistoric records are thus combined - providing data about the event sizes for which they provide the most reliable information. Latter (1995) estimated the mean interval between eruptions at New Zealand volcanos using these two records. The calculated return periods for eruptions at Mt Ruapehu are shown in Table 3-1. The eruption return periods for the other two volcanic centres considered in this risk assessment are given in Appendix A.

Much of our knowledge about volcanic processes is inferred and largely uncertain. This is the best information to date however, allowing for at least return periods within the right ball park to be estimated. There are several issues that require consideration when assigning return periods to eruptive events. These are outlined below.

The frequency of eruptions at a volcano can be dependent upon whether the volcano is in an active or dormant state. The volcano will have different eruption return periods for each state, raising the question of what data to include when trying to build up a statistical profile of volcanic history. If the volcano is presently in an eruptive period, then the inclusion of statistics from a quiet period would effectively increase the return period predicted for eruptions in the present day.

Table 3-1: Estimated intervals between eruptions at Mt. Ruapehu

Mount Ruapehu		
Erupted Volume	Return Period	Period Calculated From
$\geq 10^4 \text{ m}^3$	0.12 yrs	Extrapolation
$\geq 10^5 \text{ m}^3$	1.6 yrs	1864-1984
$\geq 10^6 \text{ m}^3$	23 ± 11 yrs	1864-1984
$\geq 10^7 \text{ m}^3$	320 yrs	1800y Before Present (B.P.)
$\geq 10^8 \text{ m}^3$	4 500 yrs	1800 yrs B.P.
$\geq 10^9 \text{ m}^3$	$63,000 \pm 31,000$ yrs	200,000 yrs B.P.
$\geq 10^{10} \text{ m}^3$	900,000 yrs (max)	Extrapolation

Another consideration is that Ruapehu's active vent has occupied several crater locations on the volcano summit over the past 10,000 years. The currently active Crater Lake vent has been the only source of eruptions for the past 2,000 years. Thus when looking at the eruptive history of Mt Ruapehu, it may be only eruptions that have occurred within this time that have relevance to the present situation at Ruapehu (Otway, 1995).

A volcano has a limited sized magma chamber. The rate at which that chamber is filled is specific to each volcano. This means that the probability of different sized eruptions is not independent, with the occurrence of a large eruption lowering the probability of further eruptions soon afterwards. The magma chamber would be at least partially emptied, and the potential for another eruption lowered. This results in the probability of volcanic events being time dependent, inferring that the volcano has a "memory" of its last eruption.

For simplification, initial volcanic modelling shall only look at the long term frequency of events.

The return periods shown in Table 3-1 are for an event greater than or equal to the magnitude stated (i.e. they are exceedence probabilities). It is not the exceedence probability that is of interest for this assessment however, but the actual return period of each event size. To limit the implied bounds of confidence in the modelling results, event magnitudes have been divided into order of magnitude groups. The return period of each order of magnitude sized event is then calculated using the following equation:

$$P(M_n \leq M \leq M_{n+1}) = P(M \geq M_n) - P(M \geq M_{n+1}) \quad (3-1)$$

where $P(M \geq M_n)$ is the inverse of the return period of M_n . Figure 3-1 illustrates this transformation, based on the data from Mt Ruapehu (Table 3-1).

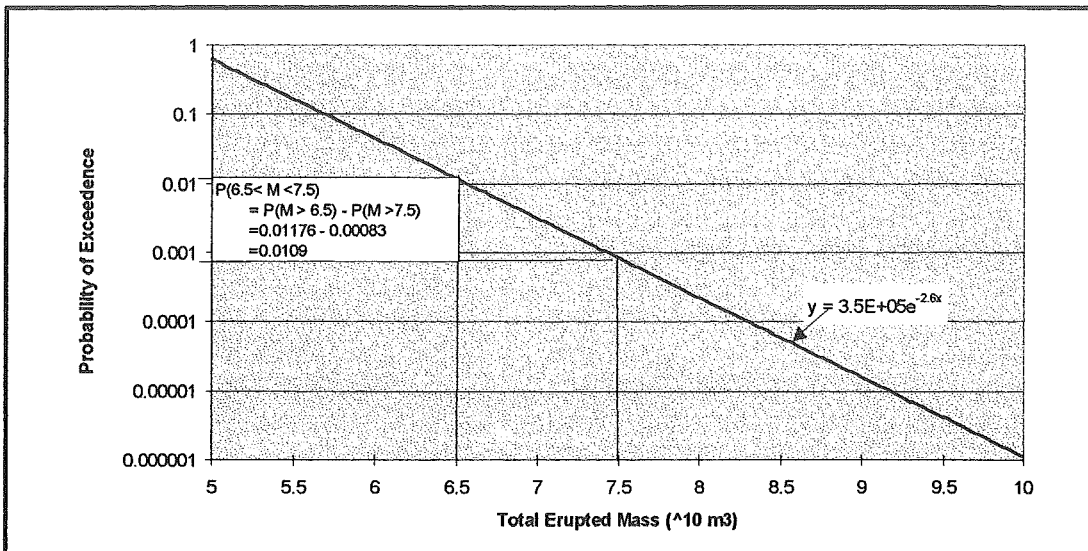


Figure 3-1: Illustration of the method used to derive actual return periods for each eruption size

As each exceedence probability has a degree of uncertainty associated with it, the actual return periods of each event size will also be uncertain. The relative magnitude of this error will be the same for both measurements (i.e. approximately 50%).

Table 3-2: Calculated return periods for eruption event sizes at Mt Ruapehu

Total Erupted Volume (m ³)	Return Period (years)
10 ⁵ (i.e.. 10 ^{4.5} - 10 ^{5.5})	0.5 ± 0.25
10 ⁶ (i.e.. 10 ^{5.5} - 10 ^{6.5})	6 ± 3
10 ⁷ (i.e.. 10 ^{6.5} - 10 ^{7.5})	90 ± 45
10 ⁸ (i.e.. 10 ^{7.5} - 10 ^{8.5})	1300 ± 650
10 ⁹ (i.e.. 10 ^{8.5} - 10 ^{9.5})	18,000 ± 9,000
10 ¹⁰ (i.e.. 10 ^{9.5} - 10 ¹⁰ _(max))	240,000 ± 120,000

3.3.2 Calculation of Ash Depths following Volcanic Activity

ASHFALL

ASHFALL is a stochastic computer model that has been developed by the Institute of Geological and Nuclear Sciences to enable rapid calculation of ashfall depths resulting from a volcanic eruption (Hurst, 1994). It was originally designed for Civil Defence use prior to or during an eruption, but is also suitable for estimating the magnitude of volcanic hazards. The program enables the calculation of ash distributions produced by a range of possible eruptions, under the various likely wind conditions.

In ASHFALL each eruption is assumed to produce an instantaneous eruption column containing a combination of ash particles and turbulent hot gases. The heat of the gases and the initial momentum of the ash particles aid the rise of the column until it

reaches a height Z_{buoy} , where the column density equals that of the surrounding air. Above this height the heavier gases lose momentum, decreasing velocities to zero at the top of the column, and finally slumping back to the equilibrium level from which it spreads laterally. The maximum concentration of volcanic gas and ash in the eruption column is at Z_{buoy} .

Once the total volume (V) of an eruption and height of the eruption cloud (Z) are known, the distribution of ash over the height of that column ($V'(z)$) can be found. The form of $V'(z)$ may be described by the Suzuki distribution (Hurst, 1994):

$$V'(z) = kV(1 - z/Z) e^{-A(1 - z/Z)} \quad (3-2)$$

where Z is the height of the eruption cloud and k is the constant of integration. The Suzuki constant A defines the relationship between Z and Z_{buoy} . A typical value of 5 is used in ASHFALL so that $Z = 0.8 Z_{\text{buoy}}$.

The amount of ash leaving the column at any height is assumed to be proportional to the ash concentration at that height. The distribution of ashfall deposits is then obtained by tracing how the ash falling out of the column is influenced by wind.

The size of volcanic ejecta ranges from fine dust to blocks. ASHFALL classes all materials with a similar settling velocity together, calculating the trajectory of each class of ash separately. During the time that an ash particle falls by one vertical step, its horizontal position will change depending upon the wind characteristics at that time. Current positions of the ash particles are calculated for each time step, until all the particles have reached the ground. The thicknesses of all the settling velocity classes are added to give the total ash thickness at points of interest on a rectangular grid.

One limitation of the ASHFALL model is that it does not model phreatomagmatic eruptions, such as would occur at Mount Ruapehu or Mount Tongariro when their crater lakes are full. The presence of lake water would change the characteristics of an eruption column from that assumed in ASHFALL. During any eruption of appreciable size, this crater lake water is likely to be ejected in the early stages of the eruption. The remainder of the eruption may then be assumed to be a Plinian type eruption.

Following a volcanic eruption, concentrations of particulates in the atmosphere may increase the likelihood of precipitation. This precipitation would result in a different distribution of ash deposition than would occur if there was no precipitation. Rain will cause a more rapid settling of the ash from the cloud, and will therefore result in errors in the suspended time for the ash. Further work could investigate the likelihood of precipitation occurring, and the impact this will have on road closure.

The input parameters required to run ASHFALL are:

- total erupted mass;
- eruption column position and height;
- fraction of ash of different sizes (and hence settling velocities);

- wind direction and velocity at each level between the ground surface and the top of the column. Wind variation with time is also required.

Total Erupted Mass

For each eruption, the total erupted volume (V) is defined as the magnitude of that event. This magnitude may then be converted into a total erupted mass (M), using a typical ash density of 800 kg/m^3 . Most volcanoes produce eruptions of a range of sizes. The full spectrum of possible eruption magnitudes shall be investigated. To limit the number of Monte Carlo simulations that are required, and also to reduce any inferred accuracy, volumes are rounded to the nearest order of magnitude.

Eruption Column Position and Height

It has been observed that the column height (Z) is positively correlated with both the eruption rate and the total size of an eruption. Carey and Sigurdsson (1989) have suggested a relationship based on observations from 45 eruptions, that may be used to estimate the eruption column height from the total erupted mass (M). This relationship is applicable for Plinian eruptions of a wide range of sizes.

$$\log_{10} M = (Z + 60.5)/7.18 \quad M \text{ in kg, } Z \text{ in km} \quad (3-3)$$

The position of an eruption column is defined using the spatial coordinates of the volcano's location.

Ash Size Distribution

An ash size distribution established from previous eruptions of the same volcano, or other volcanoes with similar characteristics can be used in ASHFALL. Figure 3-2, extracted from Woods and Bursick (1991), shows the cumulative grain-size distributions for a number of different eruptions. The Mount Vesuvius eruption (not shown in the above mentioned diagram) was of a similar nature to that which could be expected from the Central Plateau volcanoes. The grain-size distribution from this eruption was therefore used.

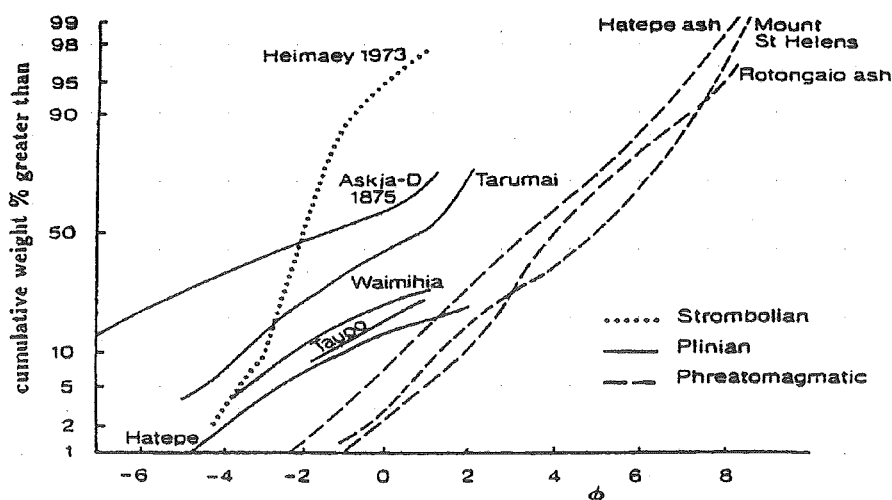


Figure 3-2: Cumulative grain-size distributions from several eruptions. Diagram taken from Woods and Bursick (1991). $\phi = -\log_2(\text{diameter in mm})$

Macedonio *et al.* (1988) estimates that 20% of all material ejected during an eruption are lithic, with a specific gravity of about 2.6. The remainder is made up of lighter particles, with a density of about half that. Combining these densities with the particle size distribution, Figure 3-3 is used to calculate the settling velocity of each particle class. The particle settling velocity distribution used to model eruptions at Mt Ruapehu, Ngauruhoe and Tongariro, is given in Table 3-3.

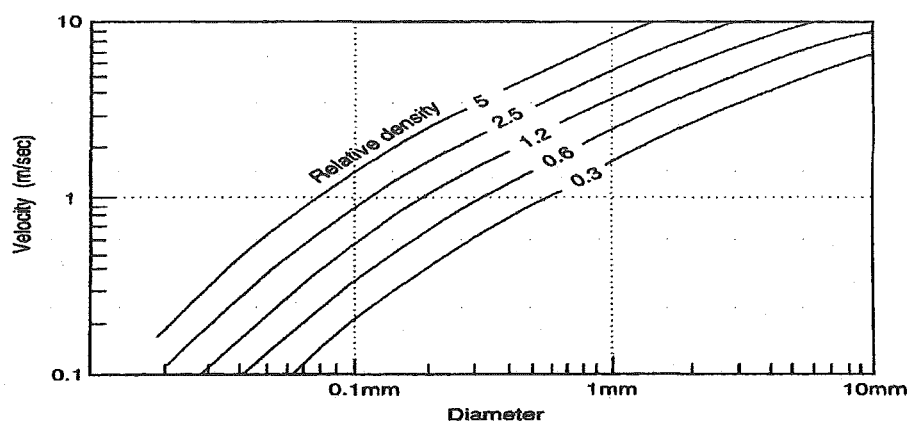


Figure 3-3: Settling (terminal) velocities of various sized volcanic ash particles. Taken from Walker *et al.* (1971)

Table 3-3: Grain size distribution used to model eruptions from each of the three volcanic centres (i.e.. Ruapehu, Tongariro, and Ngauruhoe)

Settling Velocity (m/s)	Fraction with Velocity
0.3	0.15
0.4	0.20
0.5	0.15
0.6	0.15
0.8	0.10
1.0	0.05
1.2	0.05
1.4	0.05
2.0	0.05
4.0	0.03
7.0	0.02

3.3.3 Modelling of the Wind Environment

The atmospheric environment can be difficult to model, with many different processes interacting to produce weather patterns that vary in both time and space. Traditional modelling of weather conditions focus on the prediction of future weather. In this situation however, a different approach is required. Weather records over extended periods are analysed so that the probability of different weather scenarios can be evaluated.

The ash depth distribution resulting from an eruption is dependent on the total wind profile up to the top of the ash column (which can reach 30,000 metres). Any stochastic model must therefore incorporate both the direction and velocity of the wind up to this elevation. There is a trade off to be made however, in which the complexity of the wind model should be balanced with the accuracy required of the predicted ash depths.

Description of the Wind Environment

To ensure that the model developed fits with the physical reality of the atmosphere, it is important to initially gain some understanding of the forces that drive winds. Global atmospheric circulation is extremely complex, and is beyond the scope of this study. It is possible however to identify the major factors that influence vertical wind profiles. A brief overview of each of these major factors is given.

Geostrophic Winds

The majority of wind processes are governed by the balance between the Coriolis force and the forces imposed by the horizontal pressure gradient.

The effect of the earth's rotation on any moving object, when viewed from a frame of reference that rotates with the earth, is to make it appear as though a force is acting on that object. This apparent force is called the Coriolis force (C.F). The magnitude of the Coriolis force is proportional to the velocity of the object, and acts perpendicular to its direction of travel. The Coriolis force acts to deflect objects to the left in the Southern Hemisphere and to the right in the Northern Hemisphere.

Horizontal pressure gradient forces (P.G.F) exist because of differences in air temperature and density in different regions of the globe. These pressure systems are produced by differential thermal heating of the atmosphere. Pressures in the atmosphere are expressed graphically as isobars of constant pressure.

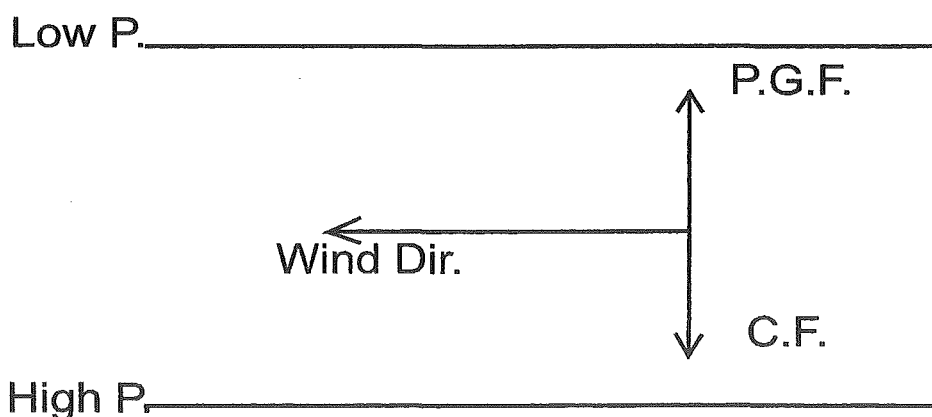


Figure 3-4: Nature of Geostrophic wind in the Southern Hemisphere

In large-scale motions in the atmosphere, winds are usually close to the velocity for which the Coriolis force and the horizontal pressure-gradient forces are in balance. When these forces are balanced, the air particles will flow parallel to the direction of the isobars (Figure 3-4). Winds in this state are called Geostrophic winds.

Gradient Winds

As evidenced by daily weather maps, typical isobars are generally not straight lines, but are curves encircling centres of high or low pressure. The circular path causes air particles to experience centrifugal force. This additional force comes from an imbalance between the Coriolis force and the pressure gradient. As the Coriolis force is directly proportional to the wind velocity, this would occur only if the wind was either faster or slower than the geostrophic velocity. A greater wind velocity, and thus stronger Coriolis force, is required for flows to curve around a centre of high pressure, and slower velocities are required for a centre of low pressure.



Figure 3-5: Illustration of the effects of Geostrophic and Gradient Winds.

Figure 3-5 illustrates the combined effects of Geostrophic and Gradient winds. The Wind direction is parallel to the isobars (dashed lines), moving with the region of low pressure (L) on the left. Wind curving tightly around the region of low pressure is moving at a slow speed, whilst the wind curving around the high pressure (H) region is moving at a faster speed.

Frictional Effects

Friction with the ground surface only affects winds up to an elevation of about one kilometre. Above one kilometre, deviations from the geostrophic or gradient winds are usually small.

Friction acts against the direction of the wind flow, slowing down the motion of the air particles. This in turn reduces the Coriolis force acting on the air particles, allowing the pressure gradient force to dominate. The wind direction is then deflected towards the region of lower pressure.

Frictional effects are strongest near the ground surface, so the general wind profile up to 1000 metres is one of increasing wind speed with height. As a result of this,

the wind direction skews to the left (in the Southern Hemisphere) with increasing elevation. The extent of this skew at ground level is typically about 5° to 10°.

Thermal Winds

Differences in vertical pressure profiles in the atmosphere are brought about by horizontal temperature variations. Pressure decreases more rapidly with elevation in colder temperatures. This means that the vertical pressure profile will be different between warmer and colder areas. This sets up a horizontal pressure gradient that changes the geostrophic wind regime. These changes are called thermal winds. An example of this is a sea breeze, which is set up by differential thermal heating over the land and the sea.

The strength of thermal winds is proportional to the gradient in average temperature, characterised by isotherms (lines of constant temperature). Thermal winds blow parallel to these isotherms. Since isotherms are generally not parallel to isobars, the direction of the thermal wind will differ from that of the geostrophic wind. To estimate wind at upper levels, the thermal wind must be added vectorially to the geostrophic wind at the lower levels (Neiburger, 1973).

Since on average, cold air lies near the poles at all levels in the troposphere (see Figure 3-6 for an illustration of the vertical layers of the atmosphere), there is a horizontal pressure gradient force directed away from the equator. This pressure gradient increases with elevation, upwards to the tropopause. Because of this, the westerly component of wind increases with height in both the Northern and Southern Hemispheres. Above the tropopause, the temperature gradient is generally in the opposite sense. The westerly component initially decreases with height and may change to easterly (McIntosh and Thom, 1969).

Available Data

The nearest available wind profile data to the Central Plateau are recorded from the Ohakea Military Air Base, just to the south of this region.

Wind conditions near ground level will be affected by features of the local terrain. As it is mainly the upper air conditions that are significant however, readings from this location may be assumed to be representative of the wind profile over the Central North Island.

Profile readings are gathered in an ongoing observation programme using balloons released from the Ohakea Air Base at six hourly intervals (NZ recording station 3206). The balloons are tracked as they rise through the atmosphere, recording atmospheric data on the way. By tracking the location of the balloon as it rises, the wind speed and direction can be established as a function of elevation.

A full year of wind profile data (three profiles for each day), recorded in 1981, was used in the development of the stochastic model. Although one year's data is not sufficient to predict the occurrence of extreme events with any certainty, it is

sufficient to characterise the general nature of the wind profile, as required for this model.

Model Development

ASHFALL requires the wind speed and direction at each elevation during the time that the ash is falling. Upper winds are often different from the lower level winds. This means that a complete profile of wind versus height up to the top of the eruption column is required. Expected changes in the wind pattern over time can also be incorporated into the model.

Simplifications and Assumptions Made

In developing a stochastic model such as this, it is essential to capture the general nature of the wind pattern. Major trends and biases must be incorporated. Smaller details, such as the exact wind direction, are not so important. Small changes in the wind direction or speed will have a minor effect on the final distribution of ash.

When deciding which variables should be included in the model, it is important to consider what the final use of the model will be. Variations in wind that bring about a vastly different ash distribution will be important in determining road closures. Examples are a significant change in ash depth or changes in the regions affected.

The direction that the wind is blowing will indicate the areas and roads that will be affected by the ash. Wind velocities are similarly important, as they dictate the distance the ash will travel. A strong wind would push ash onto communities and roads hundreds of kilometers away, whilst during calm periods the ash may be confined to the immediate vicinity of the volcano.

Major directional swings up the wind column are also important, as the spread area of the ash is substantially increased. The ash depth at any one location may be less than during an event where the wind is only blowing in one direction, but the number and length of roads affected may increase. The likelihood of this occurring needs to be predicted, as the impact of such closures will be different from an event where there is only a single wind direction.

It could be expected that the wind speed will be dependent upon the wind direction, with some directions being characterised by stronger winds. Analysis of the data indicated that variability is dominated by elevation, and wind direction at a particular elevation is of less significance. This feature has therefore not been incorporated into the model.

The ASHFALL program has the capability to include changes to the wind profile over time. This feature has also not been included, as the analysis of wind stability is complex. There will be directional dependencies, with some wind directions associated with larger currents and others with more localised, smaller weather features. As the model is presently set up, the wind profile is assumed to remain constant over the period of the eruption. This will produce a narrower, deeper band

of ash than would occur if there was a major wind change. Time dependency could be incorporated to the model at some later time if model refinement is required.

Characterisation of Directional Properties

As was discussed earlier, small changes to the wind direction are not nearly as important to the distribution of ash as large directional swings up the wind column. It is therefore this feature that dominates the modelling of the directional properties.

A directional swing was defined as a directional change of more than 60° over a 2000m elevation rise ($dD/dz = 0.03^\circ/\text{m} = 5.24 \times 10^{-4} \text{ rad/m}$). This definition was selected after inspection of typical directional change data. At locations where there is no significant change in wind direction, geostrophic winds are the driving process. Where the directional gradient (dD/dz) is greater than $5.24 \times 10^{-4} \text{ rad/m}$, it is likely that thermal winds are affecting the wind profile.

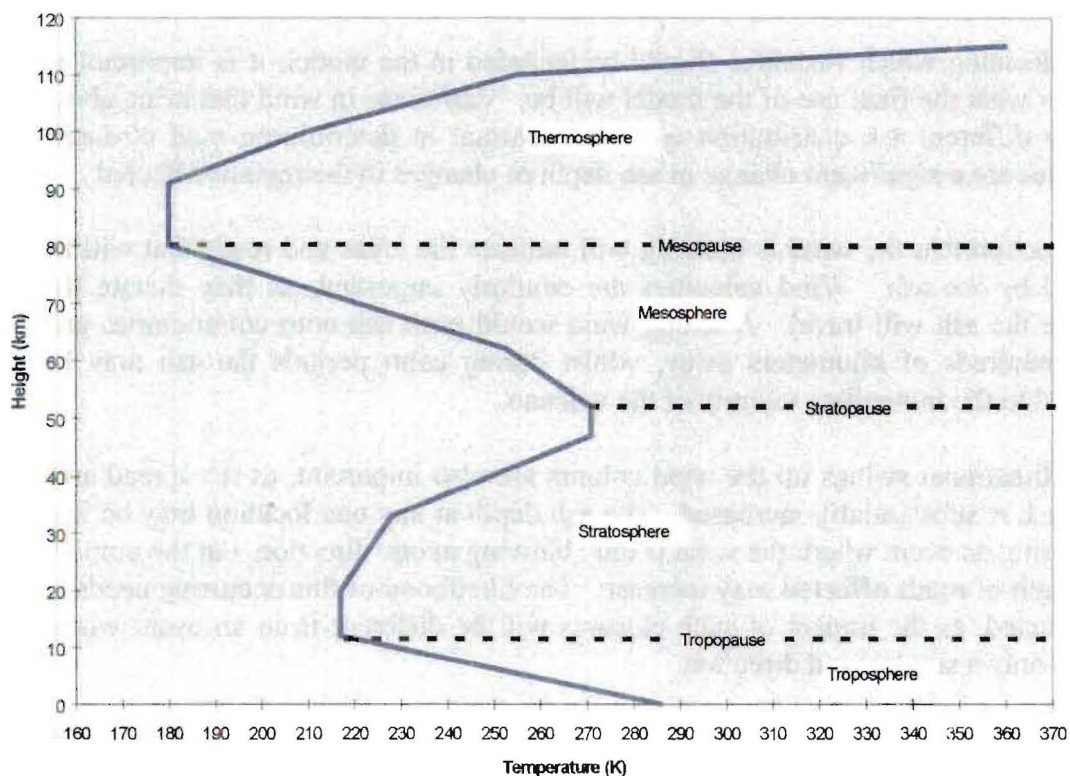


Figure 3-6: Illustration of the thermal layers in the atmosphere

The atmosphere can be divided into four layers or regions which have distinctive characteristics (Figure 3-6). In the very bottom layer of the wind column, between ground level and 2000m, frictional forces and topographic features of the landscape influence the wind environment. Frictional forces slow down the wind speed, weakening the Coriolis force. The pressure gradient then tends to deflect the flow to the right. The local terrain creates eddies and localised pockets of high and low wind speeds. This causes the wind at ground level to be quite variable, with poor correlation to the wind conditions at higher elevations. Because of this, the wind direction at ground level was modelled as independent of the wind direction at other

elevations. The distribution of observed wind directions at ground level is not characterised well by any of the standard probability distributions. Because of this the observed frequency distribution of wind directions was used in the model. It should be noted that the Ohakea wind data at ground level is unlikely to be representative of the wind conditions near the Desert Road. If a weather recording station that measures ground wind speed is set up at the Desert Road at some time in the future, then this local data could be incorporated into the model.

Between 2000 and 10,000 metres, temperature steadily decreases with elevation. This region is called the troposphere. Above this level, between 10,000 and 16,000 metres, is the upper level of the troposphere and the tropopause. The tropopause is a region where temperature ceases to vary with elevation, acting as a stable lid, inhibiting dispersive processes. Above this is the stratosphere, where temperature increases with height.

There are two distinct areas where directional swings are likely to occur. The first one is at very low elevations, particularly below 2000 metres. This is caused by localised terrain influences and frictional effects mentioned earlier. The other region of major directional change is above 16,000 metres. Here the influence of thermal winds becomes more prominent. The likelihood of a directional swing occurring at each elevation step, taken from one year's worth of observations, is shown in Figure 3-7.

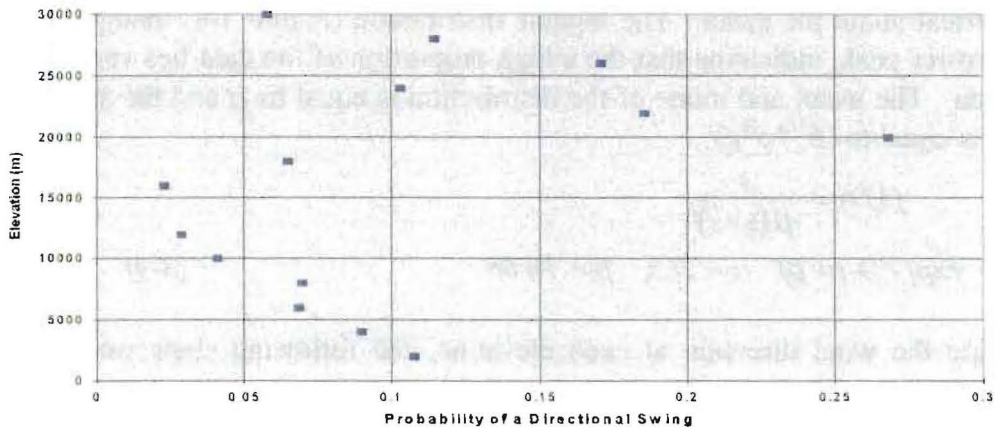


Figure 3-7: Probability of a directional swing occurring at elevations up the wind column

Depending upon whether or not a directional swing occurs, the directional gradient observed at each elevation can be described by the distributions given in Table 3-4. The distributions are different for each of the layers due to the different atmospheric characteristics impacting on wind directional changes.

Table 3-4: Distributions characterising directional change up the wind column

Atmospheric Layer	Distribution of directional gradient (dD/dz) defined in °/m	
	No directional Swing	Directional Swing (dD/dz ±0.03)
2000-10,000m	Logistic(5.82×10^{-5} , 5.56×10^{-3})	Logistic(9.26×10^{-3} , 1.82×10^{-2})
10,000-16,000m	Logistic(2.32×10^{-4} , 1.26×10^{-3})	Logistic(1.26×10^{-2} , 1.82×10^{-2})
16,000-30,000m	Logistic(-1.58×10^{-3} , 5.76×10^{-3})	Logistic(5.33×10^{-3} , 1.68×10^{-2})

A base elevation of 14,000 metres was chosen, as at this elevation there is the least amount of variability in the distribution of likely wind direction (D). Since directional data is circular, the observed directions were transformed about 270°, which is close to the average wind direction at 14,000m. This meant that the range of the data changed from:

$$0 \leq D \leq 360$$

to: $90 \leq D \leq 450$.

This was performed by simply adding 360° to any data that was less than 90°. Curve fitting could then proceed as for linear data. It was found that the wind direction at 14,000m is adequately represented by the logistic distribution shown in Equation 3-4. The logistic distribution (Owen, 1962) is similar to the normal distribution in that is symmetrical about the mean. The logistic distribution (Figure 3-8) though has a much narrower peak, indicating that the a high proportion of the data lies very close to the mean. The mean and mode of the distribution is equal to α and the standard deviation is equal to $(\beta^3 * \pi^2)/3$.

$$f(D) = \frac{z}{\beta(1+z)^2}$$

$$z = \exp[-(D-\alpha)/\beta], \quad \alpha = 263, \quad \beta = 18.68 \quad (3-4)$$

To generate the wind direction at each elevation, the following steps were then followed:

- generate wind direction at 14,000m from the logistic distribution in Equation 3-4;
- determine if there is a directional swing at any elevation (using the probabilities shown in Figure 3-6);
- generate dD/dz for each elevation from the equations given in Table 3-4;
- work both upwards and downwards from 14,000m, adding or subtracting the directional change as appropriate from the wind direction at the previous elevation.

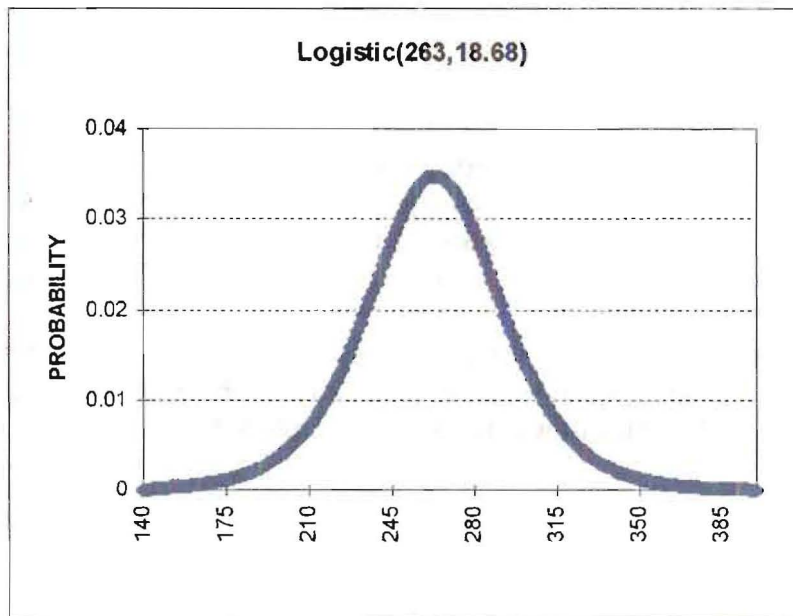


Figure 3-8: Illustration of the shape of the Logistic Distribution (263, 18,68). Note: where a direction greater than 360° is generated, the wind direction used is the generated direction minus 360° .

Characterisation of Wind Speed Properties

Characteristics of the wind speed profile also fit well into the thermal categories of the atmosphere. Wind speeds increase with elevation in the troposphere, then decrease within the tropopause. Speeds increase once again in the region of the stratosphere where temperature increases with height.

Although there is a general trend between wind speed and elevation, the wind speed profiles at any distinct time are quite varied. If there is a relatively high wind speed at one elevation, then it could be expected that the wind speed would also be relatively high at a consecutive elevation. In most instances this relationship is true. There are however times when the wind speeds appear to be quite random, with relatively high wind speeds neighbouring relatively low wind speeds.

To check the dependency of wind speeds at neighbouring elevations, the wind speeds were initially modelled as independent. To do this, speeds at each elevation were randomly generated from their respective wind speed distributions. These wind speed distributions are dependent on the elevation. The vertical gradients of velocity change (dV/dz) from the generated data were then compared to the observed velocity gradients. It was found that the generated data exhibited a significantly larger standard deviation than the observed data, indicating that the covariance between wind speeds at consecutive elevations is significant. This requires that the wind speed at different elevations to be modelled as interdependent.

To model wind speeds as interdependent, a base height of 14,000m was used. At this elevation the distribution of wind speeds (in knots) was found to fit the Weibull distribution (Equation 3-5).

$$f(V) = \alpha \beta^{-\alpha} V^{\alpha-1} \exp\left(-\left(\frac{V}{\beta}\right)^{\alpha}\right), \quad \alpha = 2.61, \quad \beta = 50.23 \quad (3-5)$$

From the observed data, the regression equation relating wind speeds at 12,000m to wind speeds at 14,000m was found. This equation can be used to find the average wind speed at 12,000m that may be expected for a given wind speed at 14,000m. Predicted wind speeds at 12,000m will be normally distributed about this mean.

This process is repeated, predicting the wind speeds at 10,000m using the 12,000m wind speed and so on up and down the wind column. The equations relating the wind speeds at consecutive elevations are shown in Table 3-5.

Table 3-5: Equations relating wind speeds at consecutive elevations (wind speed expressed in knots). Note: V_1 is the mean wind speed at Elevation₁, V_2 is the mean wind speed at Elevation₂.

Elevation ₁ →Elevation ₂	Estimation Expression.	Estimated V_2	Std Deviation of Estimate
4000→2000m	0.621* V_1 +5.53	22	8.1
6000→4000m	0.747* V_1 +3.99	27	7.4
8000→6000m	0.691* V_1 +2.80	31	10
10,000→8000m	0.749* V_1 +4.73	40	12
12,000→10,000m	0.878* V_1 +4.02	47	12
14,000→12,000m	0.938* V_1 +7.52	49	13
14,000m	V_1	45	
14,000→16,000m	0.513* V_1 +9.71	33	10
16,000→18,000m	0.809* V_1 +2.58	29	10
18,000→20,000m	0.894* V_1 +0.48	26	8.0
20,000→24,000m	0.944* V_1 +5.17	30	8.8
24,000→26,000m	1.139* V_1 +2.65	37	8.0
26,000→30,000m	1.078* V_1 +5.50	45	9.7

Model Validation

In order to assess the validity of the model developed, its predictions need to be compared to observed data to ensure realism. To check this, the data set needs to be approached from a different angle than that upon which the model was developed, finding another property that the observed data expresses. Statistical methods can then be used to see if the data generated by the model also expresses this property.

As the directional properties were modelled using the gradient of directional change, an appropriate check is to compare the generated wind directions with the observed directions at each elevation. The mean and standard deviation of the generated data (linearised as described earlier) were compared with the observed mean and standard deviation of the observed data at each elevation. It was found that there was no evidence to suggest that the wind directions generated by the model were different to

the observed data at a 99% level of significance. The variance in the predicted directions however was significantly lower than that observed in the upper part of the wind column, particularly above 20,000 metres. The model followed the general trend of increasing variance with height, but could not replicate the very large standard deviations that were observed (up to 115°). As the model attained a relatively high standard deviation though (up to 95°) it is felt that the model adequately characterises the general nature of the wind direction.

Similarly, as the model for wind speed was based on the relationships between wind speeds at different heights, the actual wind speed at each height may be used to assess the quality of the model representation. Both the mean and standard deviation of the wind speed at each elevation, for the model and observed data were compared. It was found that over the main portion of the wind profile, the wind model replicated the wind speed data well. Towards the top of the profile however, above 26,000 metres, the model started to deviate from the observed characteristics. It predicted higher wind speeds than actually occur. Only ash from the very large eruptions would reach this elevation however, so this limitation should not affect the ability of the model to predict ash distributions from most eruptions. When modelling very large eruptions, where the eruption column exceeds this elevation, predictions may not be as reliable. Ash that does reach these elevations however is likely to be carried large distances before it is deposited. This would reduce the sensitivity of Desert Road closures to these high elevation velocities.

Only one year's worth of wind data was used to develop the wind model described here. Given that this wind model was only a small part of this research (and did not prove to be an important variable), this simplification was appropriate. If the reader wanted to use this type of wind model for other analyses, where a more accurate representation was required, refinement of the model using a much larger data set (perhaps up to 10 or 20 years) would be necessary, as longer term, global weather patterns will influence the typical wind patterns over a one year period.

3.3.4 Closure Duration due to Volcanic Ash

Defining the point following a volcanic event at which the roads will require closing is very arbitrary. As shown in Figure 3-9, the factors leading to closure are both varied and interacting.

Experience in Washington after the Mount St Helens eruptions of 1980, indicated that ashfall affects traffic flow in many adverse ways. Damage to vehicles included seized engines that overheated after their cooling systems were clogged with ash, ruined vital parts that were just "worn out" by the abrasive ash, and clogged carburettors and air filters. There were also many minor road accidents in which cars ran off the road after losing traction in the ash, or smashed into unseen stalled vehicles. The fine ash was blown into clouds with each passing vehicle or gust of wind. It entered every crack and crevice of vehicles, and was seemingly impossible to control (Foxworthy, 1982).

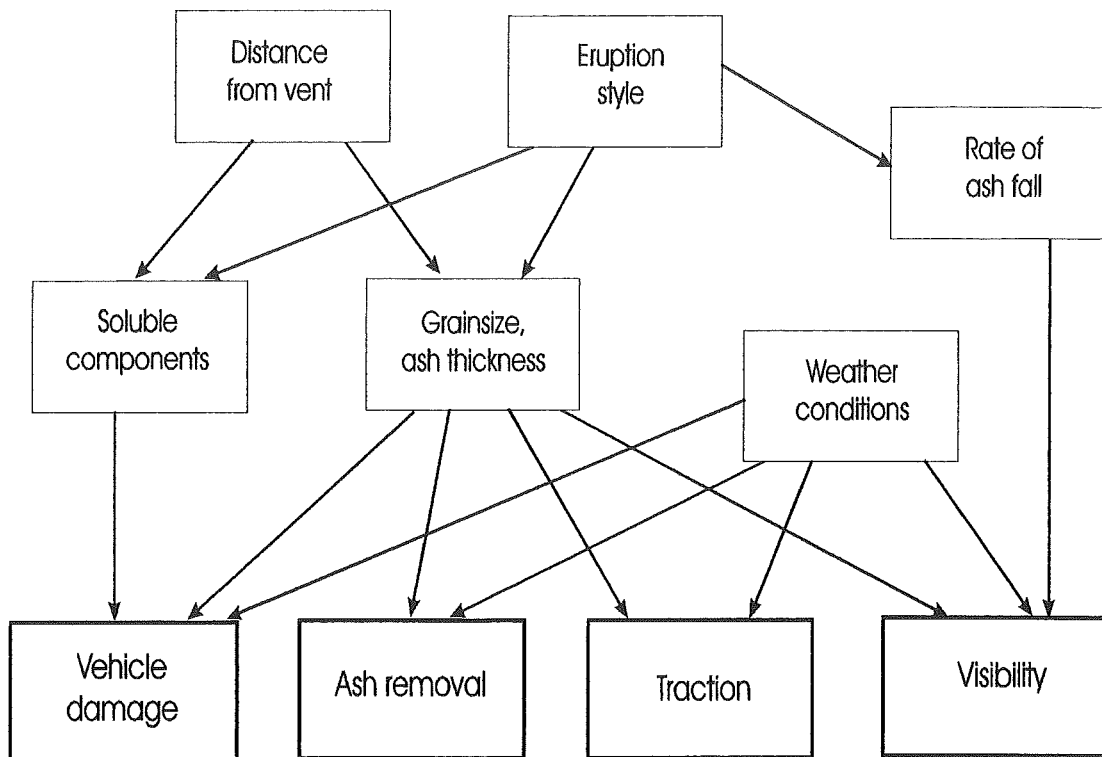


Figure 3-9: Factors affecting the impact of volcanic ash on road transport (Johnston, 1998)

Attempts were made to clear the roads using snow ploughs. Ash is difficult to remove though, as it drifts like snow but does not melt, often blowing back into cleared areas as fast as it was removed. The road clearing process was also inhibited by the snow ploughs having to stop every 100 miles or so to have ash blown from their air filters.

Aside from actually closing the roads, ashfall affected transportation links in other ways. Because of the limited visibility and potential damage to vehicles, even when roads remained open, traffic volumes were greatly decreased. To avoid accidents due to the limited visibility, and prevent faster cars from stirring up ash clouds and further reducing visibility, speed restrictions were enforced (Blong, 1984). This resulted in long traffic delays and increased journey times.

Ash depths of only a few millimetres have the potential to limit visibility. It shall therefore be assumed that nearby roads will be closed until the ash has ceased to fall during any sized eruption above a certain threshold. This assumption is supported by the experience in Washington where ash depth had little influence on the initial impacts on the transportation system.

“All areas were affected similarly during the first 48 hours of ashfall. Missoula, with 0.05 inch, was just as disabled during the first few days as Ritzville, with over three inches. More-over, the specific effects: reduced visibility, clogged filters, electrical shorts etc, were similar across all sites.” (Warrick, 1981)

Beyond this initial period, ash depths did affect the length of recovery time for transportation links in Washington, with a general trend of greater ash depths leading to longer closures. This relationship was by no means simple however, as can be seen from Figure 3-10.

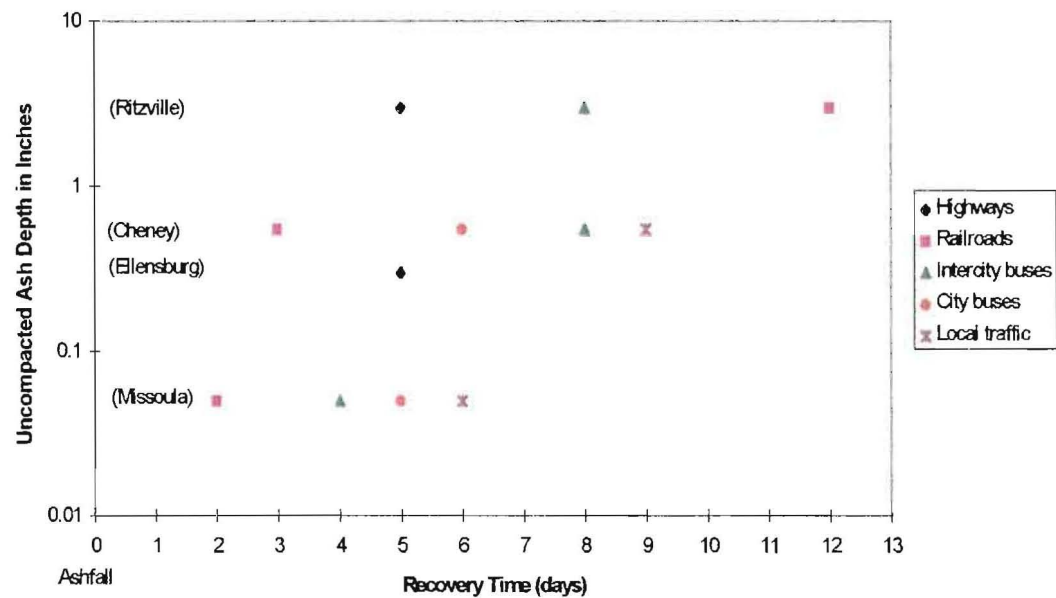


Figure 3-10: Transportation recovery times vs uncompacted ash depth, following the Mount St Helens eruption (Figure taken from Warrick, 1981)

General estimates of road closure time during an eruption are further complicated by the fact that an eruption may continue over an extended period of time. Roads will then require intermittent closure during periods of the worst ashfall.

Considering experience at Mount St Helens, estimates of closure duration and delay times that may be expected on New Zealand roads following an eruption have been made (Table 3-6). The initial estimate of risk due to volcanic ash shall only consider time when the road is actually closed, rather than the duration of general traffic disruption and delays. This definition brings the ashfall risk model into conformity with the models of the other hazards.

Figure 3-11 shows the ash depths expected on the Desert Road for a range of eruption sizes (expressed in m^3 of erupted material) at Mt Ruapehu. The variability in the ash-depths expected results from differing wind regimes that may be present at the time of the eruption.

Table 3-6: Durations of road closure and traffic impedance used in model, due to ashfall depths

Ash Depth (mm)	Road Closure Duration (days)	Traffic Restrictions Duration (days)
0 – 2	0	1-3
2 – 5	0-1	1-3
5 – 7	0-1	2-4
7 – 10	1-2	2-4
10 – 15	1-2	5-8
15 – 20	1-3	5-8
20 – 30	2-4	8-12
30 – 50	3-6	8-12
50 – 75	4-8	8-12
75 – 100	7-11	8-12
100 – 200	12-17	12-18
200 – 500	25-35	12-18
> 500	> 60	> 17

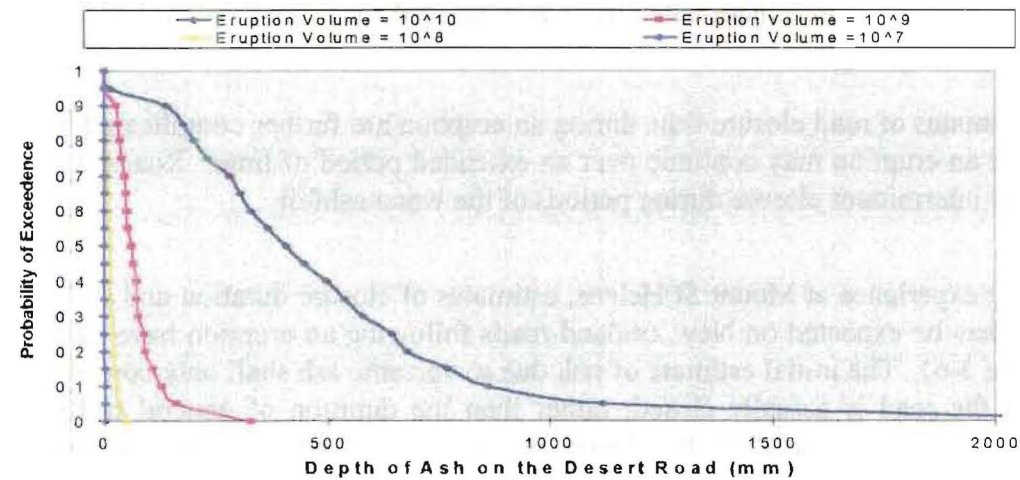


Figure 3-11: Probability distribution of ash depths expected on the Desert Road for a range of eruption sizes

3.4 Volcanic Lahar Model Development

3.4.1 Probability of Lahar Formation

As was discussed earlier, of all the volcanic centers on the Central Plateau, Mount Ruapehu is the only volcano likely to produce lahars that will affect services along the Desert Road. It is therefore only this volcano that is assessed for its potential to close the road through lahar formation.

Lahars at Ruapehu are likely to be caused by one of the following scenarios:

- ejection of the Crater Lake water onto the slopes of the volcano during an eruption;
- rapid melting of ice or snow on the cone of the volcano caused by the deposition of hot (solid or liquid) ejecta;
- the release of the Crater Lake water by collapse of the lake wall;
- sector collapse of the volcano.

One of the difficulties in assessing lahar hazard at Ruapehu is that although there is a good correlation between the size of a phreatomagmatic eruption and the corresponding lahar volume, there is no such correlation for magmatic eruptions. These may displace crater lake water, but do so gradually, generating only small lahars. An example of this type of event occurred in 1945. Lahars can also be generated without an eruptive event. The lahar that led to the Tangiwhai disaster (1953) is one such example. This lahar was caused by the collapse of a barrier of ice and unconsolidated pyroclastic materials that had impounded the crater lake at a higher than usual level (Latter, 1986).

To overcome these complications, it has been assumed that 80% of all lahars are generated by eruptive events. This is the observed ratio of eruptions to lahars that have been identified in recent prehistoric times from 1850 yr B.P. to 1861 A.D. (Hancox *et al.*, 1995). The other 20% are assumed to occur without an accompanying eruption, and thus do not occur in conjunction with ashfall effects. Where a lahar is formed during an eruption, the size of the lahar will be proportional to the size of the eruption, with an approximately similar return period (Latter, 1986).

Lahar frequencies are interpreted from lahar deposits, in much the same way that tephra deposits provided information on eruption occurrences. Hodgeson (1993) calculated return periods for various lahar magnitudes, using information provided from both the historical and prehistorical records (Table 3-7). It can be seen that return periods derived for events of the same magnitude are quite different, depending upon which record period the calculations are based. The general trend appears to be that the frequency of lahars has increased in recent times. This could be due to inaccuracies in the preservation of the lahar record. It also could indicate

the formation of the Crater Lake at about 1850 years B.P., resulting in a greater incidence of lahars as theorised by Hodgeson (1993).

The longest record is used for the calculation of the return periods of the large lahars. For smaller lahars, the most recent record is more appropriate, as this most reflects the present lahar forming conditions. In between these two extremes, where two time intervals indicate differing return periods, the return period found from the last 1,800 years is used. These return periods appear to be quite irregular, with several magnitudes having similar return periods. To find the actual return period of each lahar size, a line of best fit was drawn through the observed exceedence probabilities of the lahar sizes. Equation 3-1 was then used to calculate the actual return periods for lahars of different sizes.

Table 3-7: Recurrence intervals for lahar events at Mount Ruapehu

Ruapehu		
Lahar Volume	Recurrence Interval	Analysis Period
$\geq 10^3 \text{ m}^3$	$7 \pm 11 \text{ yrs}$	1861-1993 (n = 17)
$\geq 10^4 \text{ m}^3$	$8 \pm 12 \text{ yrs}$	1861-1993 (n = 14)
$\geq 10^5 \text{ m}^3$	$19 \pm 14 \text{ yrs}$	1861-1993 (n = 6)
$\geq 10^5 \text{ m}^3$	$120 \pm 210 \text{ yrs}$	1,800 yrs B.P. (n = 15)
$\geq 10^6 \text{ m}^3$	$29 \pm 6 \text{ yrs}$	1861-1993 (n = 4)
$\geq 10^6 \text{ m}^3$	$130 \pm 220 \text{ yrs}$	1,800 yrs B.P. (n = 13)
$\geq 10^7 \text{ m}^3$	$210 \pm 290 \text{ yrs}$	1,800 yrs B.P. (n = 6)
$\geq 10^7 \text{ m}^3$	$32,000 \pm 61,000 \text{ yrs}$	160,000 yrs B.P. (n = 5)
$\geq 10^8 \text{ m}^3$	1300 yrs	1,800 yrs B.P. (n = 1)
$\geq 10^8 \text{ m}^3$	$53,000 \pm 75,000 \text{ yrs}$	160,000 yrs B.P. (n = 3)
$\geq 10^9 \text{ m}^3$	140,000 yrs	160,000 yrs B.P. (n = 1)

It was mentioned earlier that the size of the lahar that forms during an eruption will be approximately proportional to the size of that eruption, with a similar return period. Some of the observed return periods for the lahar magnitudes (Table 3-8) do not coincide with those of the eruption return periods (Table 3-2). Where this occurred, the lahar size predicted in the model is generated randomly as either of the lahars with the next higher and lower frequency.

Table 3-8: Calculated return period for lahar event sizes at Mt Ruapehu

Total Lahar Volume (m^3)	Return Period (years)
10^4 (i.e.. $10^{3.5} - 10^{4.5}$)	6.6 ± 13
10^5 (i.e.. $10^{4.5} - 10^{5.5}$)	37 ± 74
10^6 (i.e.. $10^{5.5} - 10^{6.5}$)	200 ± 400
10^7 (i.e.. $10^{6.5} - 10^{7.5}$)	$1,100 \pm 2,200$
10^8 (i.e.. $10^{7.5} - 10^{8.5}$)	$6,200 \pm 12,000$
10^9 (i.e.. $10^{8.5} - 10^9_{(\text{max})}$)	$48,000 \pm 96,000$

Where a lahar forms independently of an eruption, it is assumed that the full range of lahar sizes are possible and that the return periods of the lahars are as shown in Table 3-8.

3.4.2 Determination of Lahar Paths

Volcanic hazard zones have been developed for Mount Ruapehu, maps of which may be found in Houghton, 1987, or Latter, 1981. Copies of these hazard maps are provided in Appendix B. The hazard maps show the paths that lahars of various sizes are likely to follow, providing an indication of the regions that will be affected and subject to damage.

The hazard maps show stylised lahar paths for events with return periods of 1-3 years, 10-30 years, 100 years, and several hundred years. Zones that would be affected by collapse of the crater wall are also shown. This event has a return period estimated between 10,000 and 100,000 years.

3.4.3 Closure Duration due to Lahars

The effect of lahar damage upon the integrity of the road network is very dependent on the individual circumstances of the eruption. The Mount St Helens eruption caused serious flooding, raised river levels, and choked water pathways with mud, logs and wreckage. These choked pathways also add the threat of sudden, catastrophic release of water impounded behind debris dams.

The likely duration of road closures due to lahars formed at Mt Ruapehu were estimated from the extent to which the lahars are expected to intercept the road network (Table 3-9). Factors such as the possibility of bridge damage, and the deposition of large quantities of mud and debris on the road were taken into account. It is unlikely that lahars with a return period of 10-30 years will impose any major damage on the State Highway bridges. A lahars with a return period of 100 years or more is expected to cause severe damage to bridges in its path.

The path of a lahar formed by an eruption will be dependent upon the predominant wind conditions. An Easterly wind would cause large lahars to flow down the Mangaturuturu Stream. A Southerly wind would push lahars into the Whakapapa and Whakapapanui catchments. The predominant Westerly wind would increase the likelihood of lahars in the Whangaehu river (Houghton *et al.*, 1987). The model incorporates the effects of wind direction on the path of lahars, and closure durations that could be expected.

The closure durations in Table 3-9 are guesstimates, and should only be viewed as such. The range in possible closure durations predicted is intended to reflect the inherent uncertainty there is in these estimates. The actual closure time due to lahars will be very much dependent upon the individual circumstances of the event.

Where a combination of ashfall and lahar debris causes the road to close, the likely closure time would be longer than if there was only one of these products causing

closure. The combined closure duration will also be less than the sum of the two independent closure times. For this assessment it was taken that the closure time for both events is the longer of the two individual closure times, plus a quarter of the other closure time. This allows for the clearing of the road of both ashfall and lahar debris to occur simultaneously, but also reflects the fact that the combined effect of the deposits will create a more difficult clearing task.

Table 3-9: Estimated closure duration of the Desert Road due to lahars generated from Mount Ruapehu

Approximate Return Period (years)	Desert Road	
	Associated Lahar Size (m ³)	Closure Duration (days)
1-3 yrs	10 ³	0
10-30 yrs	10 ⁵	0
100 yrs (Westerly winds)	10 ⁶	20-40
100 yrs (non-Westerly)	10 ⁶	15-30
Several hundred yrs (Westerly winds)	10 ⁷	120-180
Several hundred yrs (non- Westerly winds)	10 ⁷	85-125
Crater Wall Failure	10 ⁹	120-180

3.5 Probability of Closure Durations

@RISK was used to simulate the effects of a volcanic event upon the integrity of the Desert Road transportation link. For a given sized eruption or lahar event, the duration of road closure was assessed, as well as the probability that this eruption size would occur. The simulation was repeated many times, generating a range of closure frequency scenarios. These scenarios were then used to establish the frequency of closure duration relationship.

3.6 Status of Alternative Routes

Closure durations for alternative traffic routes were derived in a similar manner as that used for the duration of Desert Road closures. Ash depths along the state highways were assessed using ASHFALL, and closure durations resulting from these ash depths were calculated as for the Desert Road.

Closures due to lahars were assessed for each of the routes individually using the lahar hazard maps. Initial estimates of these closure durations are tabulated in Appendix C.

3.7 Conclusions

Modelling the effect of volcanic events on a transportation network is complicated by significant uncertainties. The effects of ash on traffic flow will be dependent upon not only the eruption characteristics, but also the meteorological conditions at the time of the eruption, and these are highly variable. This is compounded with uncertainty surrounding what the impacts of ashfall on the road network will be. The few data available are often not applicable to the New Zealand situation.

Similar problems inhibit the accurate prediction of road closures from lahars. Predictions of the probability of lahar formation, and the effects of those lahars, are limited by the availability and quality of historical data.

Given these limitations, the model developed here will provide a suitable indication as to the relative importance of volcanic events as a hazard capable of causing road closure. It provides initial estimates of the frequency and duration of any road closures that may occur, and quantifies the source and approximate magnitude of errors in this prediction.

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4. CLOSURE DUE TO EARTHQUAKES

4.1 Introduction

As understanding of the effect of earthquakes upon bridges improves, it has been acknowledged that not all existing structures will withstand the intensity of shaking for which they were originally designed. Major advances in bridge design technique took place in New Zealand in 1973, when changes were made to the general design philosophy of seismic resistance. Since that time, reinforced concrete columns in bridges have been designed to behave in a ductile manner during an earthquake. Experience shows that bridges designed prior to this time, with non-ductile detailing, are susceptible to catastrophic failure (Maffei, 1996). The effects of an earthquake upon the integrity of the road network must therefore be investigated, as there are many bridges in New Zealand that were designed prior to 1973, and are seismically vulnerable.

Seismic activity also has the potential to induce landslides. These may occur along the roadway, resulting in failed cuttings or dropouts. The location of the Desert Road on a plateau means that there are only a few areas where this risk is present. If a landslide were to occur, the duration of any road closure is likely to be relatively short compared to that if a bridge fails. It is usually possible to restore the road link for through traffic quickly, clearing at least one lane of the road. The importance of landslides caused by earthquakes thus becomes relatively insignificant compared to bridge damage, and was not investigated.

4.2 Modelling Techniques

The effect that earthquakes will have upon travel on the Desert Road will depend on the amount of damage caused to bridges along the road during an earthquake. The degree to which a bridge will be damaged will depend upon the seismic vulnerability of that bridge. Bridge vulnerability can be assessed using existing seismic retrofit prioritisation methodologies.

Finite resources require that bridges be prioritised in terms of their need for retrofit and seismic upgrade. The ranking of a particular bridge depends on three components:

- the seismic hazard of the bridge and the site;
- the vulnerability of the bridge to earthquake damage;
- and the importance of the bridge as a vital link in the transportation system (Works Consultancy, 1996).

Two methodologies for retrofit prioritisation have been developed for use in New Zealand. Each shall be briefly discussed.

4.2.1 Works Consultancy Services Limited Method

The methodology that is proposed by Works (1996) uses several stages to determine the priority of a bridge for seismic retrofit. Each stage progressively requires more detailed input, attaining a higher level of accuracy.

Prior to any detailed assessment, the bridge of interest is compared to a set of basic characteristics. These characteristics determine if the bridge meets current earthquake design standards, and can thus be eliminated from further screening procedures.

If a bridge fails the initial screening process, it is then ranked using the Preliminary Screening Process. The Preliminary Screening Process uses basic data to assign a Seismic Prioritisation Grade (SPG) to each bridge. The SPG is developed from factors relating to the bridge's importance in the transportation network, seismic vulnerability, and the level of hazard to which the bridge is exposed. This provides an initial estimate of the retrofitting priority of the bridge, with minimal time and data requirements.

Bridges with a high SPG justify more detailed seismic assessment. This would be done by an experienced Earthquake Engineer, who develops an individual "Damage vs Probability" curve for each bridge. Computer modelling is used to determine the nature of the seismic response of the bridge, analysis of which determines the damage/probability curve (Figure 4-1). The seismic unit used in this assessment is peak ground acceleration, with both the damage and probability measures expressed as a function of this.

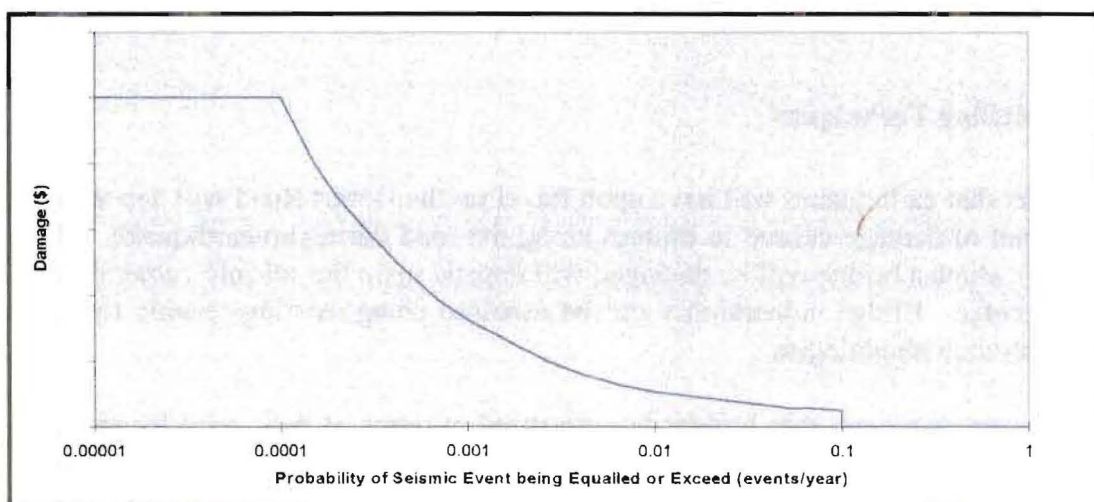


Figure 4-1: Relationship between probability of exceedence for seismic events and resultant cost of damage (Works, 1996)

Using this damage-probability relationship, combined with input from an economist, the cost incurred due to bridge failure during a seismic event is established. A cost-benefit analysis of bridge upgrading may then be conducted.

4.2.2 Maffei Method

Maffei (1996) also divides the prioritisation process into steps of increasing accuracy, with four levels of assessment recommended.

The “level zero” assessment utilises a series of flow-charts (Figures 4-2, 4-3, 4-4) to identify and assess a bridge’s vulnerable characteristics. These vulnerability characteristics are divided into three key areas:

- movement joint vulnerability;
- column/wall vulnerability;
- foundation/abutment vulnerability.

The total vulnerability of the structure is taken to be the most critical of these three components. This vulnerability is then adjusted to account for the type of soil the bridge is situated in, to give a “seismic vulnerability rating” of the bridge.

The seismic unit used in Maffei’s assessment is the Modified Mercalli Intensity (MMI) scale. The Modified Mercalli index correlates well with structural damage, and although it is a subjective parameter, it provides a valuable estimator of seismic damage. A damage/earthquake intensity curve is defined for each seismic vulnerability rating. The return periods of earthquake intensities, specific to the site of interest, are then used to establish the return period of damage to each bridge.

Maffei provides suggested relationships for calculating the cost of bridge replacement and the time required for bridge repair. These relationships may be used to quantify the costs of earthquake damage for use in a benefit-cost analysis.

Increased accuracy in the vulnerability rating estimate is achieved with higher levels of assessment. The levels proposed in Maffei (1996) are as follows:

- **Level 1 -Visual Assessment**
Field inspection and brief review of drawings. No Calculations. Consideration of seismic-force path and likely vulnerabilities. Comparison to typical earthquake performance of similar structures.
- **Level 2 -Schematic Assessment**
Level 1 plus approximate calculations. Perhaps further investigation of the soil type and structure condition.
- **Level 3 -Complete Evaluation**
More detailed engineering calculations and evaluation. Computer analysis and material testing if required.

4.2.3 Approach used in this Study

For the development of a “cost of closure” model, estimates of the closure duration-probability relationship are required. The initial estimates of this relationship need only be accurate to an order of magnitude, providing a rough indication of the relative importance of earthquakes as a closure mechanism. Once initial estimates are obtained, refinement of the closure model may then be concentrated on areas of significance.

The methodology proposed by Maffei allows for initial estimates of the closure duration/probability relationships to be made with minimum expertise, using only a level-zero assessment. Conversely, the method proposed by Works Consultancy Services relates bridge vulnerability to a closure/probability relationship only through detailed structural analysis. Thus, for initial estimates, the assessment proposed by Maffei is more appropriate.

One criticism of Maffei's method of assessment is that his initial screening procedures provide results that are too detailed, implying a degree of accuracy much greater than is actually the case (Works, 1996). Maffei's level zero assessment methodology provides conservative results, intended to highlight bridges that require more detailed assessment. It will also give a conservative and inflated estimate of the risks posed by earthquakes. If the final risk model does indicate that earthquakes are a significant closure mechanism compared to other natural hazards, then a more detailed assessment with less conservative estimates will be required. If this is the case, it may be useful to also apply Work's methodology, to check the consistency between the two approaches.

Maffei recommends initially conducting a level-one assessment on a sample of bridges to calibrate the level-zero flow chart procedure. The flow charts shown in Figures 4-2, 4-3 and 4-4, were specifically derived for the types of bridge common in rural New Zealand. For the purposes of obtaining initial estimates of vulnerability ratings, it is felt that these will not require further calibration at this stage.

4.3 Earthquake Model Development

A level-zero seismic assessment was conducted on all of the seven bridges along State Highway One, between Rangipo and Waiohuru. This assessment followed the methodology outlined below, as prescribed by Maffei (1996).

4.3.1 Assessment of Bridge Vulnerability

For each bridge along the Desert Road, the vulnerability rating of the structure (V_{STRUC}) was assessed:

$$V_{STRUC} = MAX (V_{MJ}, V_{CW}, V_{FA}) \quad (4-1)$$

V_{MJ} , V_{CW} , and V_{FA} were derived using Maffei's flow charts shown in Figures 4-2, 4-3 and 4-4.

The overall vulnerability of the bridge was then calculated as:

$$V = (V_{STRUC} \times \text{Soil Factor})^{0.85} \quad (4-2)$$

where the soil factor is as defined in Table 4-1.

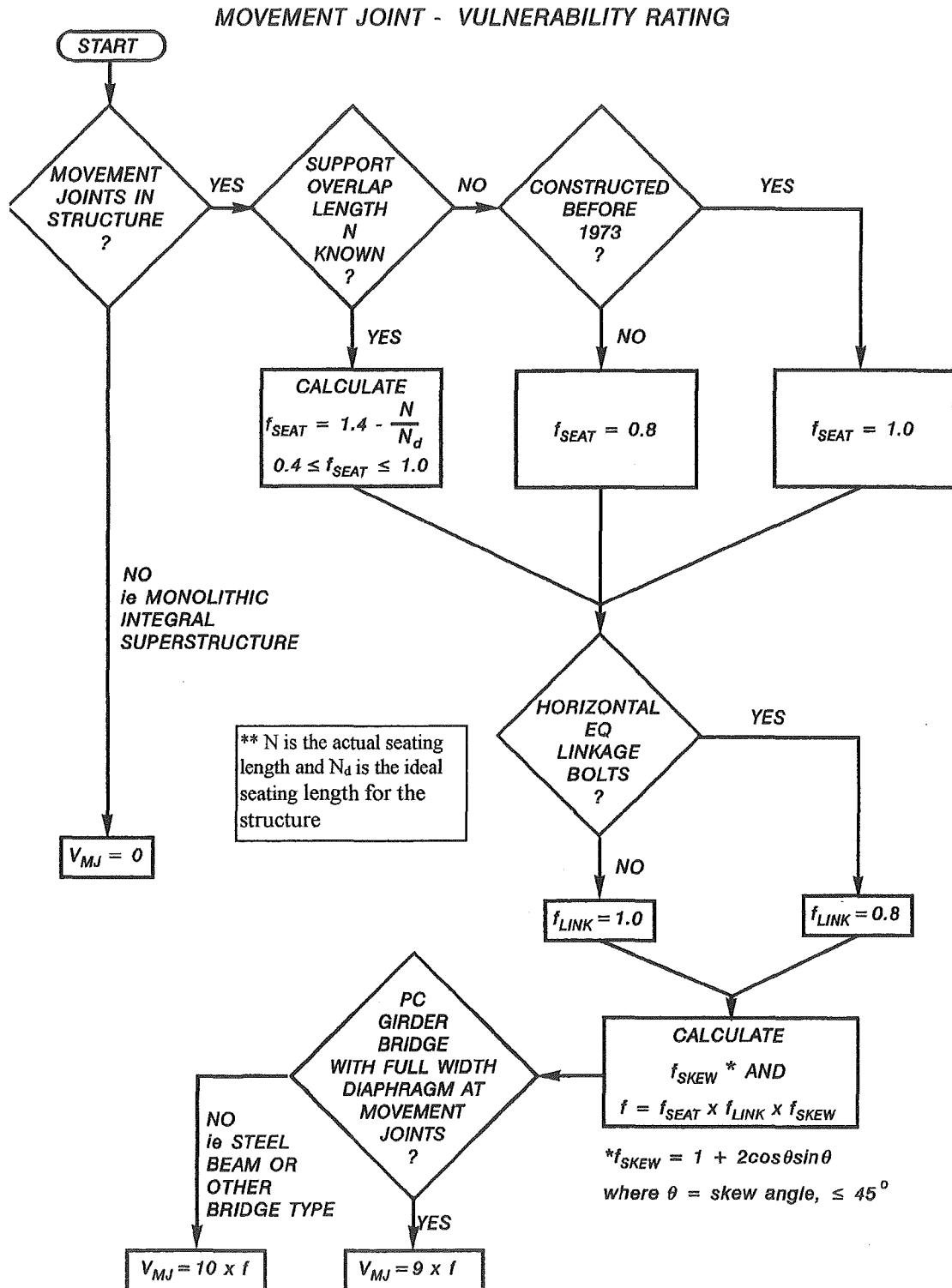


Figure 4-2: Level-zero seismic assessment flowchart for movement-joint vulnerability rating, V_{MJ} .

COLUMN / WALL - VULNERABILITY RATING

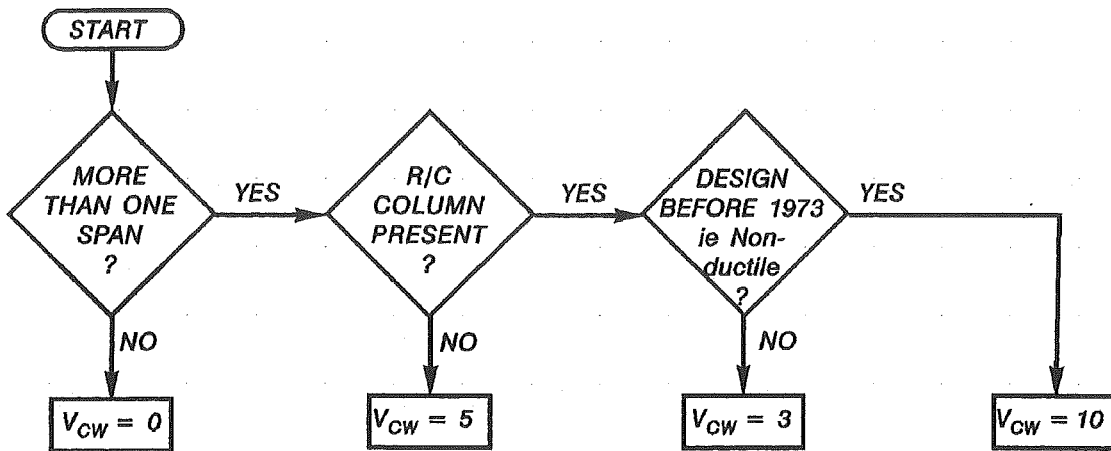


Figure 4-3: Level-zero assessment flowchart for column or pier wall vulnerability, V_{CW} .

FOUNDATION / ABUTMENTS - VULNERABILITY RATING

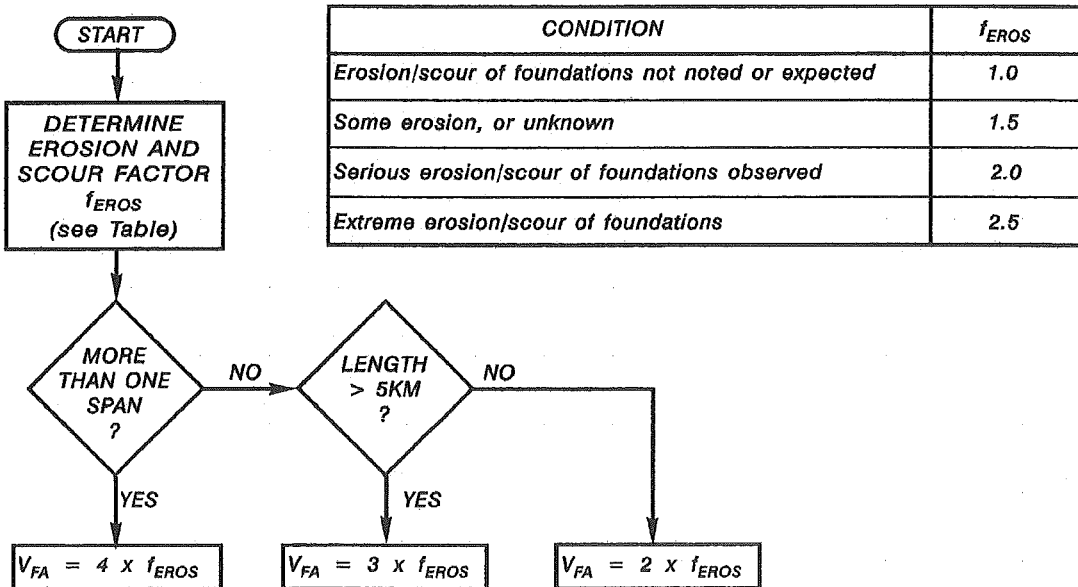


Figure 4-4: Level-zero seismic assessment flowchart for foundation and abutment vulnerability, V_{FA} .

Table 4-1: Soil Risk Factor

Soil Factor	Description
1.0	UBC soil type S ₁ or NZS 4203 Soil A (firm or rock-like)
1.2	UBC soil type S ₂
1.4	NZS 4203 Soil B
1.5	UBC soil type S ₃ or unknown soil or medium liquefaction potential
1.8	NZS 4203 Soil C
2.0	UBC soil type S ₄ (very deep clay) or high liquefaction potential

Central Plateau soils consist mainly of volcanic ash and pumice deposits. The potential for soils of this origin to liquefy was investigated by Auckland Uniservices Limited in 1996. Prior to this study, very little work had investigated the seismic response of volcanic soils, particularly with regard to the deposits found in New Zealand.

Liquefaction is a term used to describe the significant loss of strength by a saturated soil during earthquake loading. Saturated cohesionless soils, particularly sands and silts, are most prone to liquefaction. During seismic shaking, sands tend to compact causing pore pressures to be generated. Due to the relatively undrained behaviour of many sands under short term cyclic loading (such as occurs during an earthquake), this tendency for volume reduction pressurises pore water and leads to liquefaction.

Problems with volcanic sands have been encountered in the past. The Edgecumbe earthquake in 1987 resulted in widespread liquefaction of volcanic sands, and a number of failures in large geotechnical projects have occurred in the past.

In their report, Auckland Uniservices Limited (1996) defined and classified some of the soil properties of Puni River sand, which is a sand found in the Central North Island. This sand was classified as a well-graded medium to coarse sand. Its particle size distribution falls within the readily liquefiable zone defined by Tsuchida and Hayashi in 1971 (see Figure 4-5).

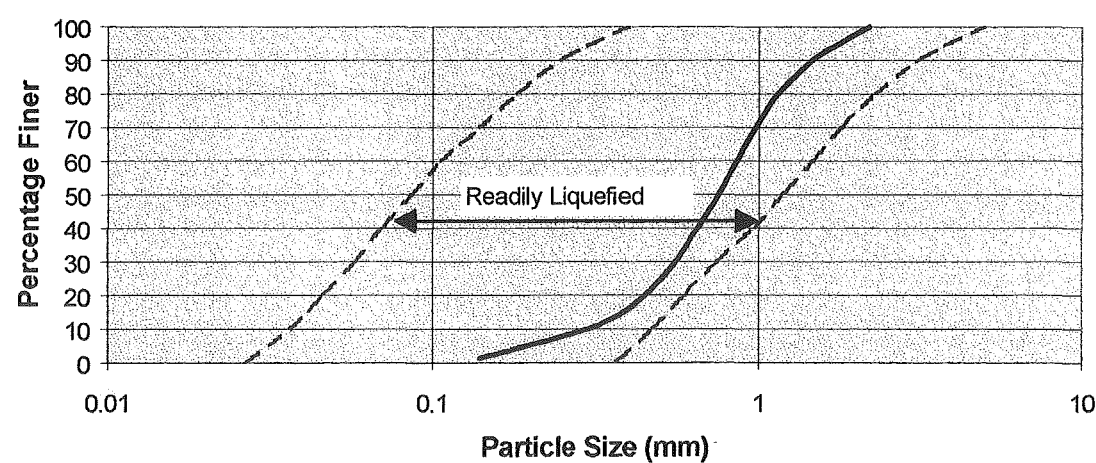


Figure 4-5: Particle Grading curve for natural Puni River sand, with liquefaction grading ranges (Auckland Uniservices Limited, 1996)

Physical testing of the Puni sand by Auckland Uniservices indicated that the liquefaction resistance curve for samples of loose Puni sands follows the well established shape of the liquefaction resistance curve for sands. This indicates that in general, the liquefaction properties of Puni sands are similar to those of other sand types. Pre-loading and consolidation of the sand sample results in a more stable sand that is more resistant to liquefaction. Dense sands are generally not susceptible to liquefaction.

Volcanic ash and clays derived from weathering are not particularly sensitive to liquefaction. Volcanic ash is more sensitive to pore water pressure increases than typical clays. This is due to its sensitive structure which can be degraded under cyclic loading. Although there will be some degradation, it is unlikely that this will be sufficient to lead to liquefaction of the soil.

Conclusions as to the seismic vulnerability of volcanic soils found in the Central North Island are unclear, and require further study. The vulnerability of bridge foundations along the Desert Road will be very dependent upon the actual soil structure, and the properties of each soil layer. For this reason, it is assumed in this model that the liquefaction potential of soils on the Central Plateau is unknown, with a soil factor of 1.5. If sands in the soil profile are present in a loose state, then this assumption may result in vulnerability estimates being too low. More extensive testing of the actual soil profile on the Central Plateau and the properties of these layers would be required before this assumption could be confirmed or disregarded.

The vulnerability ratings (V) for each of the bridges along the Desert Road, as calculated using a level-zero assessment, are shown in Table 4-2. Details of the vulnerability assessment for each of these bridges are given in Appendix D.

Table 4-2: Calculated vulnerability ratings for bridges along the Desert Road

Bridge Name	Vulnerability Rating
Poutu Canal Bridge	5.1
Puketarata Stream Bridge	6.5
Mangatawai Stream Bridge	8.4
Oturere Stream Bridge	5.1
Waihohonu Stream Bridge	10.0
Mangatoetoenui Stream Bridge	6.5
Waikato River Bridge	5.3

4.3.2 Earthquake Intensity and Closure Duration

The damage/earthquake intensity relationship for each bridge will be different depending upon the bridge's vulnerability to seismic shaking. Maffei developed a set of generic curves representing this correlation (Figure 4-6). These curves, as developed by Maffei, do not reflect the uncertainty there is about the actual level of damage there will be to a structure in any given earthquake. Damage to a structure is likely to vary depending upon the characteristics of the earthquake motion, even if the shaking intensity is the same. For this reason, it is necessary to treat the percent

damage to a structure as a random variable, with a corresponding probability distribution at each ground motion intensity (Applied Technology Center, 1985)

It has been found that the beta distribution fits observed structural damage at the different earthquake intensities well (Applied Technology Center, 1985 and Basoz *et al.*, 1995). Observed distributions of structural damage skew about the average damage level in both the positive and negative direction at different earthquake intensities. The beta distribution accommodates this characteristic well. Depending upon the relative magnitude of its two variables, the beta distribution will exhibit either positive or negative skew. The beta distribution is also bounded between 0 and 1, reflecting the limits in damage potential.

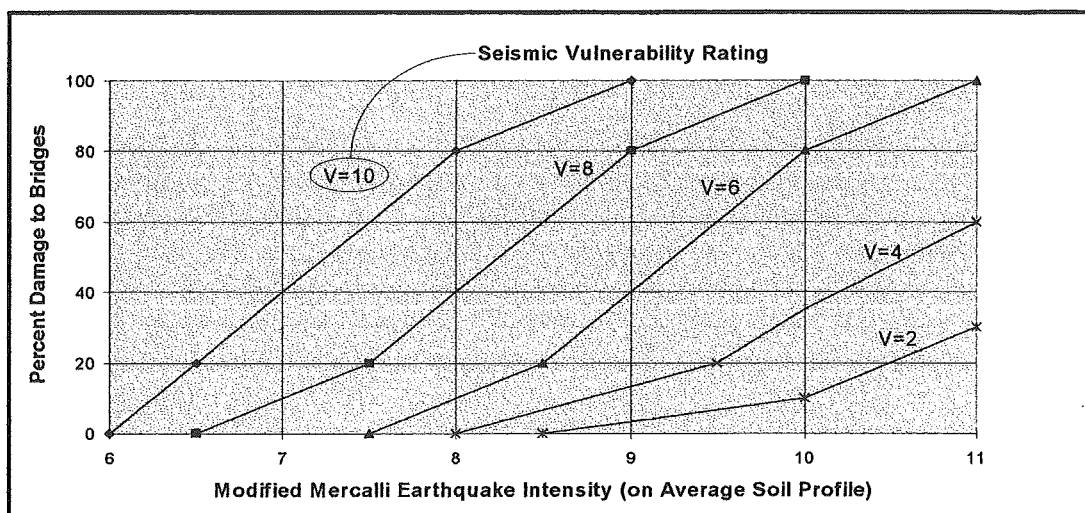


Figure 4-6: Damage versus intensity curves for different seismic vulnerability ratings

In order to find what the magnitude of the uncertainty should be at different damage levels, weighted statistics from expert opinions tabulated in Applied Technology Center (1985) were used. These statistics gave experts' opinions on their lowest, best, and highest estimates of the percent damage to a particular structure type, at different intensity levels. Data for three bridge types and a selection of other relevant structures were used. It was found that there is a good correlation between percent damage and its coefficient of variation (standard deviation/mean), with an R value of 0.91 (Equation 4-3),

$$V = 63.3e^{-0.175D} \quad (4-3)$$

where V is the coefficient of variation and D is the percent damage to the bridge.

Once the mean (μ) and the coefficient of variation (V) of the bridge damage are known, the beta parameters λ and ν may be found using Equations 4-4 and 4-5.

$$\lambda = \frac{100^2}{(0.01V)^2} - \frac{100\mu}{(0.01V)^2} - \frac{\mu}{0.01V} \quad (4-4)$$

$$\nu = \frac{100\lambda}{\mu} - \lambda \quad (4-5)$$

For each of the seven bridges along the Desert Road, a damage-intensity relationship was defined using their individual vulnerability ratings (Figure 4-6). Uncertainty, characterised by the beta distribution, was then incorporated into these relationships.

Percent damage to a bridge may be related to the length of repair time, and thus the duration of road closure. Maffei, 1996 assumes a linear relationship between the bridge replacement time and the replacement cost of the bridge.

$$\text{Replacement time} = 7 \text{ weeks} + \frac{\text{Structural Value}}{\$40\,000/\text{week}} \quad (4-6)$$

The structural value in Equation 4-6 includes the effects of terrain and bridge location upon the estimated replacement cost of a bridge. To establish structural the value of a bridge, Maffei determined a relationship between a bridge's length and its structural value, for both one and two lane bridges. There was a significant amount of scatter in the cost data used for this regression. A major source of this scatter is differing levels of reuse of old bridge components during construction of a new bridge. After an earthquake, a similar uncertainty in bridge reconstruction costs is likely. In the model, the structural value of a bridge was generated as a random variable from a normal probability distribution, with a mean predicted from Maffei's relationship, and a standard deviation of 20% of this mean (Equation 4-7). These construction costs are in 1994 dollars, and need to be adjusted to present dollar value when used in a benefit-cost analysis. The replacement time relationship however will still be directly applicable, using 1994 dollar replacement costs. The normal probability density function of cost derived for two lane bridges are as follows:

$$f(c) = \frac{1}{\sqrt{2\pi}} e^{-\frac{1}{2} \left(\frac{c - \mu_c}{\sigma_c} \right)^2} \quad (4-7)$$

where: c = construction cost (construction cost index (CCI) = 3600);

μ_c = the mean construction cost for a bridge of length L meters

= \$20,000 + \$8,000/m * L

σ_c = the standard deviation of construction costs for a bridge of length L

= 0.2 * μ_c

L = length of bridge (meters).

The bridge construction cost is then multiplied by a terrain factor. This factor ranges from 1.0 for structurally simple terrain such as short stream crossings, up to 1.3 for difficult terrain such deep ravines, or those requiring multiple span bridges. The cost is then also multiplied by a location factor, ranging from 1.0 for easy-access locations close to major towns and construction facilities, to 1.5 for remote sites where construction is more expensive.

With this information, the “percent damage versus intensity” curve can be transformed into a closure duration versus intensity relationship. Where a bridge sustains 40% damage, the repair time is defined to be 40% of the time required for total replacement of the bridge.

The equations used above estimate the replacement cost of a bridge based only on the length of the existing bridge. A more sophisticated model for estimating bridge replacement cost has been developed by Abed-Al-Rahim and Johnston (1995). This breaks the replacement cost into several component costs. Abed-Al-Rahim and Johnston’s model estimates replacement cost as the sum of the structural replacement cost, roadway improvement costs, engineering costs, and miscellaneous costs of bridge replacement. Using a set of North Carolina bridges, Abed-Al-Rahim and Johnston found that the structural cost of bridge replacement was dependent upon the length and width of the new bridge. Road improvement costs were dependent only upon the new bridge width, while miscellaneous costs were found to be proportional to the structural replacement cost, as well as being dependent upon the new bridge length. Engineering costs were proportional to the structural cost of the new bridge. All of these factors relate to the characteristics of the new bridge, details of which are often not known until the final bridge designs are completed. To overcome this, relationships were also found for estimating the new bridge length and width from the existing bridge characteristics.

The relationships developed by Abed-Al-Rahim and Johnston were established using North Carolina bridge inventory data and are thus not directly applicable to the bridges in this study. A similar approach could be used in the future to estimate bridge replacement costs in New Zealand conditions.

Temporary Access in Place of Bridge

When considering the costs incurred because of bridge damage, it is important to note that it may be possible to establish temporary access across the bridge site. This would make it unnecessary for traffic to use a longer detour route.

A temporary ford may be used across rivers where the site allows, enabling the flow of traffic to resume. This would limit the duration of road closure, and thus reduce the costs incurred from the impedance of travel. It may also be possible to construct a Bailey bridge at or near the site of the original bridge. The proximity of the Waiohuru Army Base provides a resource of equipment and expertise, making this a possible mitigation option for the Desert Road.

4.3.3 Probability of Earthquake Intensities

Site specific probabilities of earthquakes are found by identifying earthquake faults, establishing what the probable earthquake magnitudes from these faults are, calculating the distance of the site from these faults, and determining the site attenuation factors. Smith, 1986 developed a technique that defines such probabilities within New Zealand.

In Smith's model, New Zealand has been divided into regions of similar seismic characteristics. Within each region it is assumed that the seismicity is diffuse and uniform, and the law of earthquake occurrence applies:

$$\begin{aligned} \log_{10} N &= a - bM & M \leq M_{max} \\ N &= 0 & M > M_{max} \end{aligned} \quad (4-8)$$

where N is the number of earthquakes of magnitude M occurring per year within the region; a is the rate of occurrence (earthquakes/year); b is a regression coefficient; and M_{max} is the assumed maximum magnitude. The parameters a, b, and M_{max} vary from region to region.

The frequency of an intensity I earthquake occurring at the site of interest can then be determined by spatial integration, summing the frequencies from all possible fault line sources that contribute earthquake risk. The procedure may be broken up as follows.

The intensity formula is defined as:

$$I = I(obs, epi, h, M) \quad (4-9)$$

where obs is the observers location; epi is the location of the epicentre; h is the focal depth; and M is the earthquake magnitude.

This formula is then inverted to become:

$$M = M(obs, epi, h, I) \quad (4-10)$$

In practice, the complexity of the intensity formula require the inversion to be done iteratively, determining the magnitude M that will produce intensity I at the observer's location.

The seismicity model (Equation 4-8) defines the distribution of earthquake magnitudes. Integration of this gives the cumulative distribution:

$$\begin{aligned} N(M > \bar{M}) &= N_{min} \{ 10^{b(M_{min} - \bar{M})} - 10^{b(M_{min} - M_{max})} \} \\ N_{min} &= 10^a \log_{10} e / b \end{aligned} \quad (4-11)$$

where N_{min} is the total number of earthquakes of magnitude greater than or equal to the threshold M_{min} (Smith, 1986).

The parameter a (rate of earthquake occurrence) does not relate to any particular earthquake magnitude range. A new parameter a_4 is defined to be the annual number of earthquakes of magnitude 4 or greater in an area of 1000 km². The annual frequency, per 1000 km², of magnitude earthquakes M or greater is given by:

$$N(M > \bar{M}) = a_4 \{ 10^{b(4 - \bar{M})} - 10^{b(4 - M_{max})} \} \quad (4-12)$$

The annual frequency of intensity occurrence is then expressed by the integral:

$$N(I > \bar{I}) = \int N(M > \bar{M}) dA \quad (4-13)$$

where $N(M > \bar{M})$ is as defined in Equation 4-12, and \bar{M} is as given in Equation 4-10.

Derivation of earthquake intensity return periods for the Desert Road using this methodology yielded the results shown in Table 4-3. Smith estimates that errors in the predicted return period of earthquakes are of the order of 20% of the predicted return period (Smith, per. comm.). Initial modelling using a standard deviation of 20% of the predicted return period however indicated that this estimate may be too conservative, as there was distinctive grouping of the probabilities predicted for the different earthquake intensities. The final model used a standard deviation of 30% of the mean return period. The annual probability of occurrence is the inverse of the predicted return period.

Table 4-3: Return Periods for earthquake intensities within the Central North Island

Modified Mercalli Intensity (I)	Return Period (years) ± Std Dev.	Average Annual Probability of Occurrence
V	1.1 ± 0.33	0.91
VI	5.0 ± 1.5	0.20
VII	33 ± 9.9	0.030
VIII	190 ± 57	0.0053
IX	950 ± 290	0.0011

4.3.4 Probability of Closure Durations

The total duration of road closure due to an earthquake will be a function of all of the individual closure durations predicted by the closure duration/intensity relationship for each bridge. As the bridges along the Desert Road are in series, the whole road link will be non-operational until all of the bridges have been repaired, or access is in some other way restored.

In some cases, repair to bridges may be performed concurrently, reducing the total duration of road closure to the time that it takes to repair the most damaged bridge. On the other hand, access to a bridge may be limited by damage to other bridges along the road. The repair of these bridges must be completed before repair work can start on other bridges.

It is likely that the actual closure time of the road will be somewhere in between these two scenarios, depending upon the severity of damage to the access bridges. The closure duration time was assumed to be the time that it takes to repair the

bridge with the longest repair time, plus a quarter of the sum of the other bridge repair times.

$$D_c = R_1 + 1/4[\Sigma(R_2, R_3, \dots R_n)] \quad (4-14)$$

where R_1, R_2, \dots, R_n are repair times for bridges along the road network, ranked from the longest to the shortest repair duration. The closure duration/intensity relationship for the Desert Road is shown in Figure 4-7.

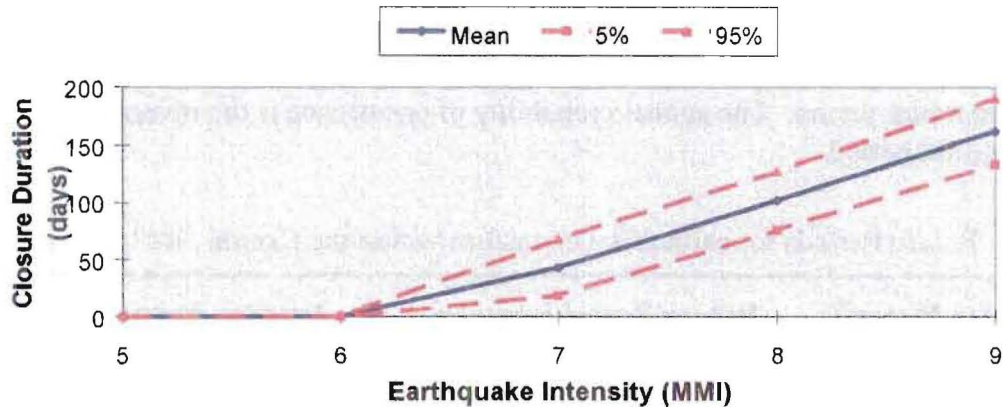


Figure 4-7: Total closure duration of the Desert Road for a given earthquake intensity.

The total closure duration for a given earthquake intensity was then combined with the probability of occurrence for that earthquake intensity, to give the probability of closure durations (Figure 4-8). Monte Carlo simulation was used to establish these scenarios.

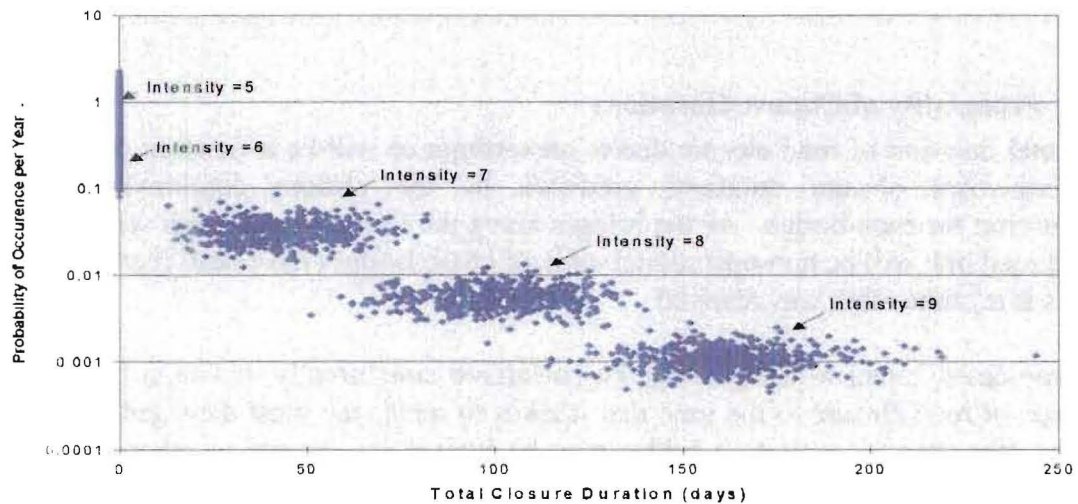


Figure 4-8: Relationship between the probability of occurrence and the duration of closure of the Desert Road for different earthquake intensities

4.3.5 Status of Alternative Routes

In order to assess the availability of alternative travel routes when bridges along the Desert Road are damaged by an earthquake, two main factors needed to be considered. Firstly, the vulnerability of bridges along these alternative routes was assessed. This was done in the same manner as the Desert Road bridges, conducting an initial level-zero assessment of each bridge's vulnerability. The calculated vulnerabilities of the bridges along the alternative routes are tabulated in Appendix E.

Secondly, the earthquake's intensity at the alternative route location needs to be considered. The distances between the alternative routes are much less than the dimensions of Smith's defined regions of similar seismicity (Smith, 1986). It may therefore be assumed that the earthquake intensity will be similar at all of the sites of interest. This assumption is supported by observed earthquake intensities (Downes, 1995), in which there appears to be no distinctive change in earthquake characteristics over the area of interest. The bridges on the Desert Road and its alternative routes were therefore assumed to be subjected to similar shaking intensities.

4.4 Conclusions

The initial estimates that have been produced by the model described here provide an indication as to the relative importance of earthquakes as a closure mechanism for the Desert Road. It should be noted though that the estimates given have been developed using only a first order assessment of the bridge vulnerabilities. The estimates will tend to be on the conservative side, over-emphasising the risks of closure due to seismic events.

4.5 References

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5. CLOSURE DUE TO TRAFFIC ACCIDENTS

5.1 Introduction

Although not a natural hazard, the frequency and random nature of traffic accidents warrant their consideration as a possible road closure mechanism.

The Desert Road is part of State Highway 1, New Zealand's major north-south road link. This means that a large number of line-haul vehicles use the road. An accident involving one of these large vehicles has the potential to block the road to through traffic, and effectively close the road link. Accidents involving multiple vehicles may also block the road, as the number of potential obstacles increases with the number of vehicles involved. The time taken to clear the road may also increase with the number of vehicles involved. Severe accidents (causing fatalities or a number of injuries) may also close the road, with the injured requiring medical attention prior to being moved.

Accidents involving trucks carrying hazardous materials are also of concern. Precautionary evacuation and thus road closure may be enforced while the security of the transported goods is checked. If there is any release of hazardous goods, clean-up operations will be necessary, and the evacuation time will increase. Where explosive or toxic goods are involved the length of road closure may be prolonged.

5.2 Modelling Techniques

To assess the frequency of road closure due to traffic accidents, the expected rate of accident occurrence must first be found. The Project Evaluation Manual (Transit New Zealand, 1991) assumes that only road improvements will affect the generation of accidents. In reality, a range of safety programs will lower the accident rate. These include enforcement, education, improvement in vehicle safety, and general road maintenance. The effect of these programs can be removed from the accident trend using a methodology proposed by Kennaird (1995). This method allows the actual accident reduction of road improvements to be evaluated.

Although accident rates are related to the density of traffic on the road, this relationship is not linear. Very high traffic densities indicate a slower travelling speed, and thus a low rate of severe accidents, though minor accidents may increase. Very low density roads have low accident rates, as interaction between vehicles is minimal, but the proportion of severe single-vehicle accidents may be high. Between these two scenarios are traffic densities that result in a high number of severe accidents.

Traffic densities along a road change with time, as daily, weekly, and seasonal variations affect traffic demand. Traffic numbers are also growing at an annual rate. To find the "base accident rate" the effects of traffic growth must be removed from

observed accident rate trends. This allows future predictions of traffic growth to be factored into the base traffic accident rate, determining the number of accidents likely to occur on that part of the road in the future.

Improved road safety management structures and focus that have been put in place since 1986 have lowered traffic accident rates. Examples are the formation of Transit New Zealand and the Land Transport Safety Authority (LTSA), introduction of the Safety Administration Programme and the National Road Safety Plan, and the setting of accident reduction targets in 1991. Extrapolation of accident trends should therefore only be done with data later than 1991. Past trends show that accident numbers for future years are inherently difficult to predict with any accuracy. Traffic safety programmes should produce a continued downward trend in accident numbers for the next 25 years, although traffic volumes and hence traffic exposure, are likely to continue to increase over this period (Kennaird, 1995).

The duration of any road closure resulting from a traffic accident on the road will be dependent upon the characteristics of that accident. Factors include the number of vehicles involved, the vehicle types, and the severity of any injuries. To estimate the duration of closures resulting from a given accident type, the personal experience of senior Fire Service officers was drawn on. This experience is based on many years of being called out to accident scenes, and will reflect the duration of road closures in the past. Recent court cases, where drivers have been charged with serious offences such as manslaughter for causing an accident, have highlighted the need for police officers to gather quality evidence at the scene of an accident. As police improve evidence gathering procedures at accident scenes, it could be expected that closure durations will increase. The actual duration increase that will be seen in the longer term however is uncertain, as an equilibrium is sought between the need to gather evidence, and public pressure to reopen the road. In these early stages of the changes, the final equilibrium level is hard to predict. Past closure durations are used in the model developed here. If future refinement of the traffic accident model is required, improved predictions of the closure durations that could be expected in the future could be included.

5.3 Accident Model Development

5.3.1 Accident Probabilities

Future predictions of accident frequency will be inaccurate unless the general trend in accident frequency is allowed for when analysing accident rates for specific sites. The underlying trend in accidents can be determined by identifying and excluding from overall accident numbers, the reduction resulting from improved roading systems. The way in which this was done is described below.

Firstly, the annual traffic growth rate was determined by fitting a linear regression line through the Average Annual Daily Traffic (AADT) on the Desert Road for the 5 years, 1991-1995 (Figure 5-1). The slope of this linear regression line is the annual arithmetic change in vehicles per year. The traffic growth rate was then found by

dividing this annual change by the predicted AADT for the succeeding year (Transit New Zealand, 1994):

$$\begin{aligned}
 \text{Traffic growth rate}_{1997} &= \text{annual arithmetic change} / \text{AADT}_{1997} \\
 &= 175 / (175 \cdot 1997 - 345965) \\
 &= 5\%
 \end{aligned}
 \tag{5-1}$$

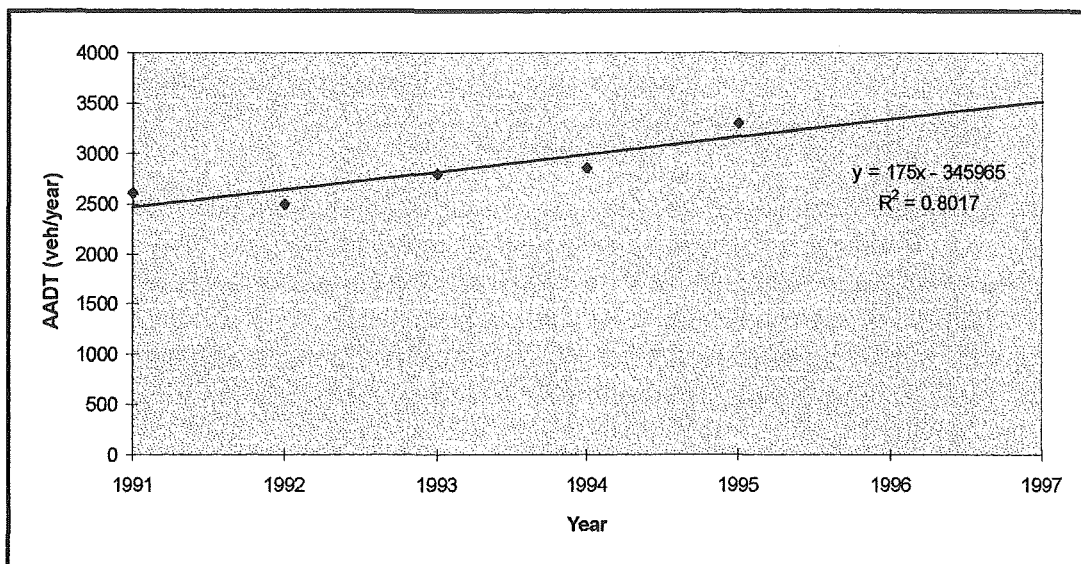


Figure 5-1: Estimation of annual arithmetic change in AADT, using linear regression of AADT's from 1991-1995

The uncertainty of the actual slope of the linear trendline, and thus the annual arithmetic change, is defined as:

$$s_B^2 = \frac{s^2}{n s_X^2} \tag{5-2}$$

where B is the slope of the linear regression line; s^2 is the variance of the data points about the linear regression line; n is the number of data points; and s_X^2 is the variance of the data along the x axis.

The predicted AADT for any year is thus uncertain, varying normally about the expected value with a variance of:

$$s_{Y_{x_0}}^2 = \frac{s^2}{n} \left[1 + \frac{(x_0 - \bar{x})^2}{s_X^2} \right] \tag{5-3}$$

where \bar{x} is the mean value of the x data; and x_0 is the x value for which y is predicted.

Combining the uncertainty of these two parameters, a distribution of possible traffic growth rates was established.

All accidents recorded on the Desert Road from 1991-1995, regardless of severity or vehicle type, were used to establish the total accident rate. It should be noted that this is different from the definition of total accident rate given in the Project Evaluation Manual (Transit New Zealand, 1991). The reason for this is that non-injury accidents have the potential to close the road, and thus have relevance to this analysis. It is likely that there is under-reporting of these non-injury accidents. This should not affect model results significantly though, as those accidents that are unreported would almost certainly not cause the road to close. One hundred and forty-five accidents were recorded over the analysis time, giving an annual accident rate of 26 accidents per year. This was assumed to be the total accident rate at the mid-point of the analysis period (i.e. 1 July, 1993).

Table 5-1 shows accident trend adjustment factors for predicting accident rates in 1997 from the accident rate established using the accident history. This accident trend adjustment factor excludes the effects of both road improvements and traffic growth. This means that if the same number of vehicles were using a section of road in 1997 as the annual number during the accident history period, the annual number of accidents expected to occur would be the annual rate during the accident history period, multiplied by the factor in Table 5.1.

Table 5-1: Factors to adjust past accident numbers to 1997 values for sites with zero traffic growth and no road improvements (Kennaird, 1995)

5 yr period of accident history	Urban (50 km/h speed limit)			Rural (70+ km/h speed limit)		
	All Vehicles	Motorcycle	Other Vehicles	All Vehicles	Motorcycle	Other Vehicles
1985-89	0.62	0.27	0.68	0.88	0.39	0.95
1986-90	0.66	0.32	0.71	0.88	0.44	0.94
1987-91	0.69	0.37	0.73	0.89	0.50	0.93
1988-92	0.73	0.43	0.76	0.91	0.57	0.94
1989-93	0.77	0.49	0.79	0.92	0.63	0.96
1990-94	0.78	0.54	0.81	0.95	0.65	0.97
1991-95	0.83	0.61	0.85	0.97	0.71	0.99

Accident trend adjustment factors for use at a particular site must allow for the actual traffic growth at the site. The accident trend adjustment factor that should be used is the appropriate factor from Table 5-1, multiplied by the ratio of the traffic volume at the future time of interest (1997) to the traffic volume at the mid-point of the accident history.

$$\begin{aligned}
 &\text{Accident trend adjustment factor} && (5-4) \\
 &= (\text{factor from Table 5-1}) * (AADT_{1 \text{ July, } 1997} / AADT_{1 \text{ July, } 1993}) \\
 &= (\text{factor from Table 5-1}) * ((\text{Traffic growth rate} * \text{No of years}) + 1)
 \end{aligned}$$

Whilst it is reasonable to use this linear extrapolation of accident trends over a short period, it would be unreasonable to expect such a trend to continue over 25 years. This is especially true if the trend is a significant annual reduction or increase in accidents. An exponential trend that asymptotes to zero change is more reasonable

for discounting accident reduction benefits over this longer period. An exponential reduction was achieved using an equivalent arithmetic growth rate (Table 5-2), which is dependent only on the accident trend adjustment factor. The arithmetic growth rate factor dictates the shape of the exponential curve.

Table 5-2: Equivalent arithmetic growth rates for use in discounting accident costs (Kennaird, 1995)

Accident trend adjustment factor	0.6	0.7	0.8	0.9	1.0	1.1	1.2	1.3
Arithmetic growth rate for accidents	-7.6%	-6.2%	-4.4%	-2.4%	0.0%	2.6%	5.5%	8.6%

The final model only uses one average annual cost of accidents, and does not incorporate the effects of traffic growth on both the cost of closures and the frequency of accidents over a 25 year period.

It was assumed that the proportion of each accident type (i.e. severity and type of vehicle involved) making up this total accident rate will remain similar to that observed during the accident analysis period of 1991-1995 (Table 5-3).

Table 5-3: Proportion of accidents between 1991 and 1995 falling into each accident severity and vehicle type classification

Vehicle Involvement	Accident Severity		
	Non Injury	Minor or Serious Injury	Fatality
Single Car ¹	0.255	0.234	0.014
Multiple Car	0.090	0.145	0.028
Single Truck	0.055	0.048	0.000 ²
Multiple Truck ³	0.062	0.055	0.014

5.3.2 Duration of Road Closure after an Accident

The duration of a road closure can be broken up into several major components. There is the travel time for emergency vehicles to reach the scene of the accident, and then time for the injured to be medically examined, extracted from the vehicles, and removed from the scene of the accident. Particularly where more than one vehicle is involved, the police will be wanting to investigate the accident cause. This requires a preliminary screening evaluation of the accident scene. The road is marked and sprayed prior to any debris removal so that evidence may be preserved. The road then needs to be cleared of debris and obstacles before traffic flow can resume (Golob *et al.*, 1987).

¹ Where "Car" is taken to be all vehicles excluding trucks and buses.

² The true proportion of this type of accident occurring is unlikely to be zero. In the model, a very small proportion was assigned to this accident type.

³ Where "Multiple Truck" refers to any accident involving a truck and one or more other vehicle.

The duration of any closure will depend on which emergency service is requested by the persons reporting the accident. If only an ambulance is requested then it is unlikely that the road will be formally closed. This measure is not seen as a priority for the attending ambulance officers. If the fire department is requested, then generally they will be assisted by an ambulance. The fire department will readily close the road, particularly because of the type of accidents to which they are called. If police presence is requested, then they may be assisted by an ambulance or the fire department as required. The police have the power to close the road, and will do so if necessary. If at all possible, both the fire and police departments will strive to leave one remaining lane open for the travelling public when closing the road. For this study, only times when both operational lanes of the road are closed, effectively severing the transportation link, were assessed.

Every accident will be different, with individual scene characteristics affecting the duration of resulting closure. Factors independent of the accident type, such as weather conditions, will also affect closure times. For example, snow and ice conditions will require extra precautions to be taken, resulting in longer closure times. Public pressure will also affect the duration of road closure. Closures longer than about six hours attract a lot of public attention, and pressure mounts on emergency services to reopen the traffic route. For this reason, only in exceptional circumstances will roads be closed for longer than this period (Price and Shields, 1997).

An accident involving only one car is unlikely to fully close the road, as it is usually possible to allow one lane to remain operational. An accident involving more than one vehicle, causing injury or death, will require an accident investigation. This will extend the closure time beyond that needed for emergency response and treatment of the injured.

Accidents involving trucks are very likely to close the road, primarily due to the bulky nature of trucks, increasing their propensity to obstruct the road. Closure durations for accidents involving trucks are far less dependent on accident severity than for car accidents. This is because removal of the truck governs closure durations, as opposed to the requirement of medical treatment. It is also likely that the freight the truck is carrying will need to be unloaded. If the truck is carrying fluids, then these will need to be decanted.

It is estimated that approximately 90% of all trucks carry some form of hazardous material. Of these trucks, 20% will be carrying sufficient quantities of hazardous material to create a major problem following an accident. Once it is noted by the attending emergency services that a truck is carrying hazardous material, it will generally take about an hour to assess the implications of this risk. Where it is found that the hazardous materials are causing major risk, a specialist hazardous materials team may be requested. These delays may add an additional 1 to 4 hours to closure times.

All of the following estimates of road closure probabilities and durations (Table 5-4) were established after discussion with people experienced in accident emergency

response (Price and Shields, 1997). As was discussed earlier, recent changes in procedures followed when gathering evidence for criminal convictions, means that future closure durations may differ from those seen in the past. Future model refinement could include obtaining more current estimates of closure durations. Estimates could also be improved by cross matching accident occurrences with the records of the fire, ambulance, and police departments. Using information on who was called out, and the length of time they were at the scene, closure times can be assessed. This would allow for closure durations to be statistically analysed.

Table 5-4: Estimated probabilities of closure $P(C)$, and closure durations D_c , for each severity and vehicle involvement accident type, where D_c is the estimated average closure duration.

Vehicle Involvement	Accident Severity		
	Non-injury	Injury (Minor & Serious)	Fatal
Single Car	$P(C) = 0$	$P(C) = 0.1$ $D_c = 15 \text{ min}$ $D_c = 0 - 30 \text{ min}$	$P(C) = 0.1$ $D_c = 15 \text{ min}$ $D_c = 0 - 30 \text{ min}$
Multiple Car	$P(C) = 0.5$ $D_c = 0 - 15 \text{ min}$	$P(C) = 1$ $D_c = 1 \text{ hr}$ $D_c = 0.5 - 2 \text{ hrs}$	$P(C) = 1$ $D_c = 1 \text{ hr}$ $D_c = 0.5 - 2 \text{ hrs}$
Single Truck	$P(C) = 0.9$ $D_c = 2 \text{ hours}$ $D_c = 1 - 4 \text{ hours}$	$P(C) = 0.9$ $D_c = 2 \text{ hours}$ $D_c = 1 - 4 \text{ hours}$	$P(C) = 0.9$ $D_c = 2 \text{ hours}$ $D_c = 1 - 4 \text{ hours}$
Multiple Truck	$P(C) = 1$ $D_c = 4.5 \text{ hrs}$ $D_c = 2 - 6 \text{ hrs}$	$P(C) = 1$ $D_c = 4.5 \text{ hrs}$ $D_c = 2 - 6 \text{ hrs}$	$P(C) = 1$ $D_c = 4.5 \text{ hrs}$ $D_c = 2 - 6 \text{ hrs}$

5.3.3 Probability of Road Closures

@RISK was used to simulate the effects of traffic accidents upon the integrity of the Desert Road transportation link. For a given accident severity, the duration of road closure was predicted. This closure duration was then paired with a probability of occurrence for this event, to give a return period of the road closure duration. The simulation was repeated many times, generating a range of closure frequency scenarios. These scenarios were then used to establish the frequency of closure duration relationship.

5.3.4 Status of Alternative Routes

Closure of the Desert Road due to an accident requires that traffic divert to an alternative route. This would increase the traffic density on that alternative route. Because accident occurrence is related to the density of traffic on the road, the probabilities of accidents occurring on both the Desert Road and its alternative routes are not statistically independent. Closure of the Desert Road will alter the probability of an alternative route also closing due to an accident.

As the scope of this analysis is to look at the risk of Desert Road closures, closure of alternative routes are only of interest when they coincide with Desert Road closure. When the Desert Road is closed, the traffic that would have been using the Desert Road will divert to the alternative routes. The traffic behaviour model that is described in Chapter 7, predicts the number of vehicles that will divert, and the routes that they will take. It is assumed that the traffic accident rate on the alternative routes (expressed in accidents per vehicle kilometre) will be the same as that observed on the Desert Road. This assumption is a simplification, as each road will have its own characteristics and accident black spots, that will affect the rate at which accidents occur on the road. As traffic accidents are not a major closure mechanism, this simplification is acceptable. The type of accidents to occur on the alternative routes are also assumed to be similar to that observed on the Desert Road. The type of accidents to occur will be dependent upon the characteristics of the road, but will also depend on the traffic composition. As it is the Desert Road traffic that is diverting, it is reasonable to assume that the type of accidents to occur will be similar.

The probability of both the Desert Road and one of its alternative routes being consecutively closed will be the product of the probability of the Desert Road being closed and the probability of the alternative route closing.

To simplify the analysis, the effect of only the Desert Road and one other alternative route closing simultaneously was assessed. It could be expected that if one of the alternative routes was closed, the traffic demand on remaining alternative routes would be increased. This would further increase the probability of an accident on these alternative roads. Conversely, it is also possible that traffic density would reach a point where traffic is slowed, and the accident rate is decreased. The probability of more than two routes being concurrently closed by traffic accidents however is sufficiently remote compared to other hazards that excluding this scenario from the risk of closure model is not significant.

5.4 Conclusions

Future accident rates are difficult to predict as they are a function of many complex factors. The method used here should provide reasonable estimates of the average accident rate that can be expected. The actual number of accidents to occur in a year will vary about this average, since accidents are a random occurrence.

Traffic accidents are a relatively frequent event, so information as to how long the road will be forced to close is easier to obtain than for other hazards. Experience of officials who respond to accidents was drawn on to make predictions of the length of road closure that can be expected. These predictions should provide a reasonable interpretation of the past closure durations that have occurred. A question remains as to whether or not the future duration of closures will follow these estimates. Future refinement of the model could include updated estimates of future closure durations.

5.5 References

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6. INTERDEPENDENCY OF CLOSURE MECHANISMS

6.1 Introduction

In previous chapters, it was assumed that the hazards assessed occur randomly in time, independently of each other. This allowed road closures to be modelled separately for each of the hazards, assuming that only one hazard will occur at any particular time. In this chapter, the applicability of these assumptions is assessed and, where necessary, the degree of interdependence is quantified. Figure 6-1 shows the relationships that could exist.

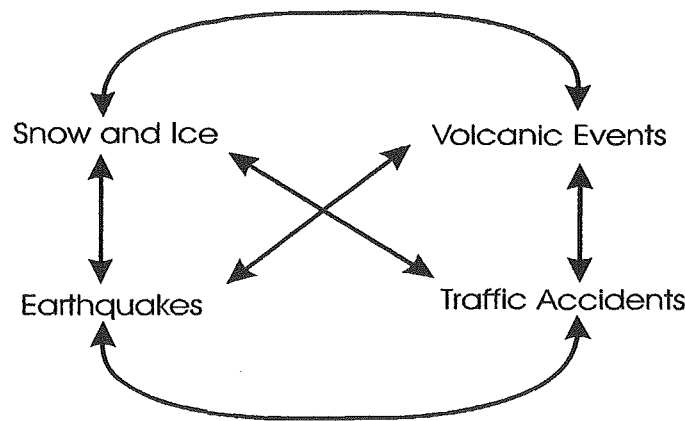


Figure 6-1: The interdependencies that may exist between the hazard types

For there to be independence between hazard types, the occurrence of one type of hazard must have no effect on the probability of another type of hazard occurring. If two hazards are related in some way, then this assumption will not be true. For example, the presence of snow and ice on the road may worsen driving conditions and increase the probability of a traffic accident occurring. Conversely though, the presence of a traffic accident will have no effect on the probability of snow and ice forming on the road. Thus the occurrence of an accident will be correlated to the presence of snow and ice on the road, but the presence of snow and ice on the road is independent of an accident occurring.

If two hazard events were to occur concurrently, then the duration of road closure will be a function of the closure durations expected if the hazard events were to occur independently, but the resulting closure duration will not necessarily be the sum of these two closure durations. Returning to the example of snow and ice and a traffic accident occurring concurrently, the duration of road closure may be longer than if only one of the hazards were present. The time taken for the accident to clear may extend beyond the duration of closure due to snow and ice. The total duration though is likely to be less than the sum of the closure durations attributed to each hazard event. In many cases, snow clearing operations may proceed at the same time as the accident is being cleared, thus shortening to the total duration of closure. The actual combination of closure times will be uncertain, and is complicated by the individual circumstances of each closure scenario.

6.2 Analysis of Relationships

In the following section, interdependencies between each of the hazard types are explored and, where possible, the probability of two hazard events occurring concurrently and the resulting closure durations are evaluated.

6.2.1 Snow and Ice / Volcanic Events

The presence of snow and ice on a road has no effect on the probability of a volcanic eruption occurring. Meteorological conditions have no known triggering effect for volcanic processes. The presence of snow and ice though, may heighten some of the effects of an eruption. When there is snow and ice present on the Desert Road, it is reasonable to assume that general weather conditions in the area will be poor and that there may be fresh snow on Mount Ruapehu. This snow will contribute to lahars that are formed on the mountain. Any additional fresh snow attributable to a single storm event though, is unlikely to increase a lahar size by an order of magnitude, which is the level of accuracy of this study. Also, the presence of snow on the mountain is not primarily dependent on snow or ice being present on the Desert Road, nor is the total depth of the snow pack dependent solely on weather conditions at a particular time. It has therefore been assumed that snow and ice conditions will not significantly alter the size of any lahar, or the duration of road closure that would result.

There is some evidence that ash ejected during very large eruptions may alter global climate regimes. Volcanic aerosols ejected into the atmosphere create a stratospheric haze, blocking incoming solar radiation and causing a general cooling at the earth's surface. This impact is complicated by the natural variability in global temperatures, particularly with El Niño and Southern Oscillation (ENSO) effects altering average temperatures over time. Observations following past eruptions indicate that although there is a general global cooling following an eruption, winters in some areas may actually be warmer. It has been suggested that this warming is due to enhanced zonal winds that are driven by heating in the tropical stratosphere by the volcanic aerosols (Robock and Mao, 1992).

Only very large volcanic eruptions have the potential to significantly alter global climates in the manner described above, and are certainly outside the scope of this study. On a more local scale however, there may be some immediate impacts on weather that may affect road closures. These changes occur in the first few days following an eruption. Mass and Robock (1982) studied weather records from the North-Western part of the United States for the few days immediately following the Mount St Helens eruption in 1980. They found that where the ash plume from the eruption arrived at an area during the daytime, it blocked solar insolation. This resulted in nearly steady temperatures for several hours, reducing daily maximum temperatures. At night the plume worked to suppress infrared cooling, holding night-time temperatures higher than normal. This resulted in a much smaller diurnal temperature range than normal. These effects disappeared almost completely within two days of the eruption (Mass and Robock, 1992).

The experience after the Mount St Helens eruption would suggest that immediately following an eruption at one of the Central Plateau volcanos, conditions along the Desert Road would be cooler during the day, but remain relatively mild at night. As a majority of icing is initiated in the early hours of the morning, this would mean that an eruption is likely to inhibit ice from forming. Because of this, closures due to snow and ice and a volcanic eruption would not be expected to occur concurrently. These local climatic effects of an eruption are expected to clear within a couple of days, so that any impact will be short term and not of great significance to this study. The relationship between volcanic eruptions and weather patterns was therefore not investigated further.

6.2.2 Snow and Ice / Earthquakes

There is no known relationship between the probabilities of certain meteorological conditions occurring and seismic activity. There is no reason why the presence of snow and ice should amplify or reduce the damage incurred by bridges during an earthquake. Neither should an earthquake extend the duration of snow and ice presence on the road surface. These two hazards are therefore independent.

6.2.3 Snow and Ice / Traffic Accidents

The likelihood of snow and ice forming is not dependent upon whether or not a traffic accident has occurred. Conversely, the likelihood of a traffic accident will be dependent upon whether ice is present on the road. The relationship between the two is complicated by a range of conflicting factors, as described in Palutikof (1991).

“There is no simple relationship between road accidents and weather. In certain circumstances, people drive more slowly and carefully and, where possible, postpone or cancel their journeys. This leads to a reduction in the total number of accidents and in the number of serious accidents per unit distance travelled. In wet conditions, conversely, the number of accidents increases. The situation is complicated by a host of other factors which include the longer hours of darkness in winter and the greater volume of traffic on certain days of the week and at certain times of the day.”

Changes to drivers' willingness to travel during winter storm conditions were also highlighted in a study by Hanbali and Kuemmel (1993). They showed that the average reduction in traffic volumes due to winter storm conditions depends directly on the severity of the weather conditions, with more severe storms resulting in greater reductions in traffic volume.

Forbes and Katz (1957) studied the causes of winter accidents. They found that although accidents on dry roads amount to a substantial proportion of all accidents, when the probability of an accident is expressed in terms of exposure (distance travelled), the proportion of accidents that occur on icy roads becomes significant.

Passenger car drivers are generally responsible for accidents in icy conditions at a higher rate than truck drivers (Forbes and Katz, 1957). This is primarily due to passenger car drivers' failure to cope with the road conditions. Fixed object collisions were the most common type of accident, followed by rear-end and

sideswipe accidents. Estimates of travelling speeds at the time of accidents showed that drivers do considerably reduce their speed, but still misjudge the icy driving conditions (Logan, 1992).

Of the 145 accidents that occurred on the Desert Road from 1991 to 1995, 23 were at least partially attributed to ice or snow conditions on the pavement surface. This is 16 percent of all accidents that occurred. Statistical analysis was used to find if the type of accidents caused by snow and ice differed from the observed characteristics of all accidents on the road (such as single vs multiple vehicle accidents and car vs truck accidents).

Equation 6.1 is the probability density function (pdf) describing the value of a proportion p of some particular event occurring in a collection of events. For example p might represent the proportion of all accidents attributable to ice that are single car accidents. If there are n accidents attributable to ice, and of these m are single car accidents, then the best estimate of p is $\hat{p} = m/n$. The pdf describing p is equation 6.1. Equation 6.2 gives the standard deviation of p . This standard deviation decreases with the square root of the total number of observations.

$$f(p) = \frac{1}{\sqrt{2\pi\sigma_{\hat{p}}^2}} \exp\left(-\frac{(\hat{p}-p)^2}{2\sigma_{\hat{p}}^2}\right) \quad (6-1)$$

$$\sigma_{\hat{p}} = \sqrt{\frac{\hat{p}(1-\hat{p})}{n}} \quad (6-2)$$

The proportion of all accidents where ice was not a contributory factor that fall within a certain category, such as single car accidents \hat{p}_1 is then compared to the proportion of accidents where ice was a factor fall within the same category, single car accidents \hat{p}_2 . The null hypothesis is that there should be no significant difference between these two proportions and that the likelihood of a single car accident occurring is independent of whether or not ice is present. For this null hypothesis to be not true, the statistic z (Equation 6-3) must be significantly different from zero and have a low probability of occurring. The value of z indicates the likelihood that the difference in the observed proportions is just due to statistical variability, or if the proportions were drawn from populations with different characteristics. The statistic z represents a normally distributed variable with a zero mean and a unit variance.

$$z = \frac{\hat{p}_1 - \hat{p}_2}{\sigma_{\hat{p}_1 - \hat{p}_2}} \quad (6-3)$$

$$\sigma_{\hat{p}_1 - \hat{p}_2} = \left[pq \frac{n_1 + n_2}{n_1 n_2} \right]^{1/2} \quad (6-4)$$

where \hat{p}_1 is the proportion of accidents falling into a given category where ice is not a contributory factor, and \hat{p}_2 is the proportion of accidents fitting the given category

where ice is a contributory factor. The standard deviation of the difference of the two proportions is found using Equation 6-4 where p is the proportion of all accidents falling into the given category (such as the proportion of all accidents that are single car accidents), and q is equal to $1-p$.

Three different accident characteristics were tested. Firstly, the severity of accidents occurring under icy conditions and non-icy conditions were compared to see if they significantly differ. It was found that accidents in icy conditions are less likely to cause serious injury or fatality than other accidents. There is no evidence though at a 5% level of significance that this observed difference is significant.

Similarly, there was no evidence to suggest that cars are more likely to be involved in an accident caused by icy conditions than trucks. This finding appears to contradict the observation by Forbes and Katz that passenger car drivers misjudge icy conditions more often than truck drivers. This is probably due to the increased exposure of trucks to icy conditions. Ice normally forms on the road during the night. At this time, car traffic flow is low, but the number of trucks traveling is relatively high. Thus the propensity for car drivers to misjudge icy conditions is countered by their lesser exposure to the hazard. Improved car technology since 1957, such as radial tyres fitted to cars, will also have altered how cars respond to icy conditions.

The proportion of accidents involving only one vehicle caused by ice was then compared to the proportion of multiple vehicle accidents that involved icy conditions. It was found that at a 5% level of significance that there is enough evidence to reject the null hypothesis and suggest that single vehicle accidents are more likely to have ice as a contributing factor than multiple vehicle accidents. This means that when estimating the proportion of accidents that are caused by the presence of ice, it is necessary to differentiate between whether the accident involves only one or multiple vehicles.

There will be uncertainty about the actual proportion of accidents that can be attributed to ice. Estimates of the true proportion can be expected to follow a normal distribution as described in Equation 6-1. The proportion of single vehicle accidents with ice as a contributing factor has a mean of 0.22 and a standard deviation of 0.04. The proportion of multiple vehicle accidents occurring in icy conditions was found to have a mean of 0.07, and a standard deviation of 0.03.

Estimating the proportion of accidents that have ice as a contributory factor is useful in predicting savings in accident costs that can be attributed to anti-icing measures. Mitigation options that prevent ice from forming on the road will also presumably prevent those accidents that have ice as a major contributing factor from occurring.

Closures due to snow and ice impose a significantly higher annual cost to the New Zealand economy than do traffic accidents. Any accident that is caused prior to the road being closed is likely to be attended to and cleared concurrently with the snow and ice clearing procedures. In some instances, the clearing of an accident may extend the duration of the snow and ice closure. This could happen if a truck accident needs to be cleared from the road, but is delayed until the snow and ice has been at least partially removed. This additional closure duration is unlikely to be

significant however, as the extra duration will be relatively small and occur only rarely.

6.2.4 Volcanic Events / Earthquakes

Volcanic eruptions and seismic tremors are strongly related. It is very common for volcanic activity to be preceded or accompanied by earthquakes, or more generally, by seismic vibrations (Schick, 1988). These earthquakes are generated by pressure fluctuations associated with degassing processes of magma within the volcano (Schick, 1992).

With more than one volcano on the Central Plateau, each with its own characteristics, earthquakes generated in the area do not fit easily into standard classifications. Latter (1981) developed a classification methodology specifically for these volcanoes, with earthquakes being defined as either “volcanic”, “volcanic-tectonic”, or “tectonic” events. A tectonic earthquake is defined as one that has taken place in competent rock by some instantaneous source mechanism. Earthquakes with a volcanic origin are those which occur in heat-weakened or partially molten material by some extended source mechanism. Those earthquakes of a tectonic nature, but which occur very close to a volcanic center, are volcanic-tectonic events.

Typically, during the early stages of an eruption at Mount Ruapehu, there is a one for one relationship between individual earthquakes and the discrete eruptions making up an eruption sequence. Later however, after the major energy release at the surface has taken place, low-frequency earthquakes become almost continuous, while the explosions at the surface give way to uninterrupted steam emission. The magnitudes of these earthquakes are usually small, rarely exceeding magnitude 5 on the Richter scale. They are typically much less damaging than the associated eruption (Johnston, 1998). At Ruapehu, the largest low-frequency volcanic earthquake yet recorded was a magnitude 4.0 earthquake in 1945 (Latter, 1981).

Whether or not an earthquake is an indicator of imminent volcanic activity appears to be dependent upon whether or not the volcano’s vent is in an “open” or “closed” state. Eruptions have been seen to accompany small, low-frequency earthquakes (Magnitude=2.0-2.7) when lake temperatures are high, the high lake temperatures indicating that the vent is open. Conversely, there have been other times when relatively large, low-frequency earthquakes (Magnitude=3.4) have not been accompanied by an eruption. During these times there have also been uncharacteristically low temperatures in the crater lake, indicating that the vent is closed.

There is still much uncertainty and debate as to the exact relationship between volcanic-earthquakes and eruptions. Given however the widespread damage to the road network that could be expected following an eruption sequence due to ash and lahars, it is not expected that the additional damage that may occur to bridges due to earthquakes induced by the eruption will be significant. Where bridge damage is widespread, the return period of damaging earthquakes associated with volcanic events are expected to be longer than the return period of tectonic earthquakes of a similar magnitude. There is likely to be no additional benefit in retrofitting for volcanic earthquakes over and above that from retrofitting for tectonic earthquakes.

Besides this primary correlation between volcanic eruptions and seismic events, there is also the possibility that a tectonic earthquake may trigger instability in the Crater Lake wall of Mount Ruapehu. If the Crater Lake wall were to fail whilst the lake level was high, this could lead to devastating lahars moving down the Whangaehu catchment. Hancox *et al.* (1997) determined that this scenario is unlikely. Failure of the wall below the level of the filled lake is not expected even with shaking intensity up to Modified Mercalli Intensity 8. This possibility was therefore not considered.

6.2.5 Volcanic Events / Traffic Accidents

The presence of a traffic accident on the Desert Road would have no impact on the probability of a volcanic event occurring. A volcanic event though could affect the traffic accident rate, as ash fallout from the eruption sequence impairs driving conditions on nearby roads. Ash limits visibility, reduces traction, and can cause vehicles to break down or stall due to blocked air filters in the motor. All of these factors would increase the likelihood of an accident occurring. Conversely, drivers are likely to adjust their driving speeds to meet the conditions, reducing the severity of accidents that may result. Also, similar to snow events, knowledge of the road conditions and the threat it may pose, can actually deter people from making trips. This would lower the traffic flow on the road, reducing the exposure to accident situations.

The road closure durations predicted to occur following a volcanic eruption take into account the increased probability of accidents occurring, and the disruption to traffic flow that these would cause. It is therefore not necessary to calculate a correlation between the probability of an accident occurring and the presence of volcanic ash. This relationship has already been incorporated into the ashfall model. If a mitigation option were to be considered that would reduce the probability of accidents occurring following an eruption, then the number of accidents that could be prevented would need to be estimated. Unfortunately there are few effective mitigation options that reduce the impacts of a volcanic eruption. Ploughing ash off the road surface may speed the resumption of traffic flow, though increased traffic speeds may increase the probability of accidents, as it is likely that the road conditions would still be less than ideal.

6.2.6 Earthquakes / Traffic Accidents

The occurrence of a traffic accident will have no effect on the probability of an earthquake occurring. During an earthquake, severe shaking could cause some steering difficulties, resulting in accidents in extreme cases. An earthquake of this magnitude is likely to also result in significant bridge damage. Additional closure times due to accidents on the road are likely to fade in significance when compared to the closure durations caused by this bridge damage. Driver caution following an earthquake should lessen the probability of road accidents due to damaged road surfaces or bridges. Again, repair time to bridges and the roadway is likely to exceed the road closure duration due to vehicle accidents. In summary, the effect of an increased likelihood of traffic accidents both during and following a seismic event is expected to be minimal.

6.3 Conclusions

Table 6-1 summarises the interactions that may exist between the hazards that can cause closure of the Desert Road. The ticks in this interaction matrix indicate that there is “some” relationship between the hazards, though as discussed, many of these co-dependencies are not significant to this study. The only co-dependency relationship that is likely to be significant for modelling purposes is that between the probability of snow and ice on the road surface, and the likelihood of a traffic accident occurring. The snow and ice model has been modified to account for the joint probability of these two hazards.

Table 6-1: Interaction matrix of causative mechanisms for road closure

		Responsive			
		Snow and Ice	Volcanic Events	Earthquakes	Traffic Accidents
Causatives	Snow and Ice		√		√
	Volcanic Events	√		√	√
	Earthquakes		√		√
	Traffic Accidents				

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7. ASSESSING THE RISK OF ROAD CLOSURE

7.1 Introduction

The amount it is worth spending to prevent closures of the Desert Road and its alternative routes will depend on how much the closures actually cost the New Zealand economy. The cost of road closures will be dependent upon the degree of disruption that the road closures impose on travel. Closure of the road will impact upon some trips through the Central North Island more than others. Where there are alternative routes readily available, the increased cost of travel along this detour may not be large. The cost of other trips however may be significantly increased, sometimes even to the point where it is no longer worthwhile to make the trip at all. To evaluate the degree to which closure of one or more of the road links will effect travel, a model of the road network in the Central North Island was developed. The details and rationale behind this model are described in this chapter.

7.2 Definition of Closure Risk

Risk is a function of both the probability of a hazard event occurring, and the consequences of that event. So far in the context of this study, the consequences of the hazard events have been expressed in terms of the duration of road closures that result. Some hazard events have the potential to close more than one road link at any one time, increasing the impact of the hazard. The consequences of an event can be expressed in terms of the closure duration of the Desert Road, the closure duration of State Highway 49, etc. In order to make all of the closure scenarios comparable, it is necessary to express the consequences of closure in common units. This is done by expressing the impacts of the closures in terms of the cost imposed on the New Zealand economy.

The costs imposed on the New Zealand economy because of any closures, will be dependent on how travellers respond to the reduction in route options. Travellers will always use the route that they perceive has the minimum cost associated with it (this phenomenon is further explained in section 7.3.3). This implies that when all routes are available for use, travellers will choose a route that minimises their costs. When some of the road links in the network are closed, trips then have to be redistributed to other routes through the network. As the new routes are different from those initially chosen, the cost of travelling on the new routes must be more than the cost of travelling on the original route. The additional travel cost to each road user is then the difference between the cost of travel when links are closed, minus the cost of the trip if all roads were open. The cost to the New Zealand economy will be the sum of these differentials for all travellers on the network (Equation 7-1).

$$C_T = \sum_a C_{aF} F_{aF} - \sum_a C_{aI} F_{aI} \quad (7-1)$$

where C_T is the travel cost of the road closure, c_{aF} is the cost of travelling on link a when a road is closed, c_{aI} is the cost of travelling on link a when all links are open, and F_a is the flow on link a for each of the scenarios.

7.3 Development of a Traffic Model for the Central North Island

The first stage in finding the cost of road closures, is to gain an understanding of how traffic behaves within the road network under normal conditions, when all road links are open. In order to do this, a model was developed of the Central North Island traffic network. This traffic model was then used to predict traffic behaviour when one or more of the road links are closed.

SATURN (Simulation and Assignment of Traffic to Urban Road Networks), a computer model developed by the Institute for Transport Studies at the University of Leeds, was used to simulate traffic behaviour. SATURN requires two main types of input. The first one includes details of the road network. The road network is specified as a series of nodes (population centres or road intersections) and connecting links (the roads). It also requires an origin-destination matrix for the network, detailing where people travelling in the network are coming from and going to. The model uses this information to determine how each traveller will pass through the network, which routes they will select, and how much the trip will cost. The detailed workings of this model are explained in the next sections.

7.3.1 Representation of the Road Network

In SATURN there are two types of networks that can be employed; a simulation network and a buffer network. The simulation network is the more detailed of the two, describing the properties of each intersection within the network, allowing delays and traffic interaction to be modelled. This type of network is useful for predicting traffic behaviour within congested urban networks, particularly delays to turning traffic, and the build up and depletion of traffic queues. The other type of network that can be specified in SATURN, the buffer network, is a less detailed representation of the road network. Details of the road links, such as the length and maximum travel speed, are included but there is no input specifying intersection characteristics. This type of model is useful for uncongested rural roads, or areas of urban modelling where accurate intersection analysis is not required. In representing the road network of the Central part of the North Island, only a buffer network was used.

A buffer network is made up of a number of nodes, connected together by links. Links are the roads modelled in the network, and nodes are either points at the intersection of two road links, or population centres where trips are produced from or attracted to. The buffer network that was developed for this study is shown in Appendix F. It should be noted that not all roads in the area are represented; only those that were thought to be significant in the general traffic flow through the area

were included. All of the links and nodes are numbered, and are always referred to in SATURN by that number.

Information input for each of the links include:

- the nodes that bound it on either end;
- the link distance;
- the link flow capacity;
- the travel time along the route under free flow conditions;
- the travel time or speed of travel at capacity conditions;
- a parameter indicating the shape of the cost-flow curve for that link.

For more details on the way a buffer network was developed, and the rational behind the network representation used in this study, the reader is referred to Moustafa Omar (1998).

7.3.2 Prediction of the Origin-Destination Matrix for Travellers on the Network

Any trip made within the network will have an origin and a destination (i.e. where the trip began and ended). Defining the origin-destination (OD) matrix for a network is a way of classifying all trips in the network by where they are coming from and going to. The study area is divided into sub-areas or zones. The OD matrix has each zone listed down the side of the matrix as possible origins, and also along the top of the matrix, where they are representing possible trip destinations. The number of trips going from Zone B to Zone C (T_{BC}), is shown in the box that lies at the intersection of the Zone B row, and the Zone C column (Table 7.1). Note that trips with their origin and destination as the same zone (i.e. those lying on the diagonal of the OD matrix) are intra-zonal trips, which are essentially local trips within the zone. The number of these intra-zonal trips is assumed to be zero for this study.

Table 7-1: Layout of an Origin-Destination (OD) Matrix

O-D MATRIX		Destinations		
		Zone A	Zone B	Zone C
Origins	Zone A	T_{AA}	T_{AB}	T_{AC}
	Zone B	T_{BA}	T_{BB}	T_{BC}
	Zone C	T_{CA}	T_{CB}	T_{CC}

Zones are connected to the network, so that trips between zones can be made using the links and nodes (roads and road connections) comprising the network. The total number of trips originating from a particular zone is said to be the “production” of that zone. The total number of trips travelling to a zone is the “attraction” of that zone.

The number of trips produced and attracted to each zone in the study area will depend on the features of that zone. These include characteristics such as the land use in the area, dwelling density, number of employers working within the area, etc. The number of features affecting trip generation and production, and their interdependencies, compound to make it difficult to predict the number of trips originating and terminating at each zone.

Traditionally, to determine the origin-destination (OD) matrix for a network, surveys of peoples' travel characteristics had to be conducted. This involved either stopping traffic at selected roadside locations, and requesting information as to where the motorist was coming from and heading to, or surveying households about their trip making on a particular day. Both of these methods have significant disadvantages in that they are expensive to conduct, disruptive, slow to analyse, and inaccurate. These studies are also cumbersome in nature, and hence people are reluctant to conduct them. Because of this, they are also invariably out of date.

To overcome these problems, SATURN uses a method known as "ME2" or "matrix estimation using maximum entropy". ME2 uses traffic count data to find the most likely OD matrix that would produce the flows observed on each road link. This type of approach is attractive, as traffic count data are relatively easy and cheap to obtain, and there is already a substantial amount of this type of data gathered.

Traffic counts can be seen as the result of combining a trip matrix and a route choice pattern. The route choice pattern will be dependent upon how trips are assigned to a network. This is explained in more detail in Section 7.3.3, but very briefly, traffic may be assumed to either follow an all-or-nothing assignment to routes, or a more complicated equilibrium assignment.

All-or-nothing assignment assumes that all travellers starting from the same origin and going to the same destination will travel on the one route. This means that the proportion of vehicles using the cheapest route will equal 1.0, with no vehicles using the alternative routes.

Equilibrium assignment acknowledges that as the number of vehicles travelling on a route increases, so too will the cost of using that route. Thus at some point, it will be cheaper for some of the vehicles to travel on an alternative route. The proportion of vehicles using each route equates to the route choice proportion (p_{ij}), where i is the zone from which the trip originates, and j is the destination of the trip of interest.

Denoting the flow on link a by V_a , the fundamental equation in the estimation of a trip matrix from traffic counts is:

$$V_a = \sum_i \sum_j p_{ija} T_{ij} \quad (7-2)$$

where p_{ija} is the proportion of trips from i to j using link a . We work with link flows rather than route flows because link flows can be observed. Route flows on the other hand cannot be observed readily, as routes involve a multitude of links, which will also form parts of other routes through the network. Initially it shall be assumed that

the proportion of trips using each link (p_{ija} factors) are known. Providing there are a sufficient number of traffic counts available, the OD matrix values (T_{ij}) may be determined uniquely by solving simultaneously Equation 7.2 for all links. In most study areas however, the number of unknown T_{ij} 's (i.e. the square of the number of nodes) will either be more than the total number of links in the network, or the number of available independent traffic flow counts. This means that the problem is under-specified, and the values of T_{ij} cannot be determined uniquely.

As the solution is under-specified, there will be a number of possible OD matrices that satisfy the observed traffic count data. To determine which matrix is the most appropriate, additional information as to the likely proportions of the elements of the OD matrix are used. This could be an OD matrix for the network that has been already estimated, or one that was previously found from travel surveys. The ME2 program selects the matrix that predicts the least variation from this prior matrix. Where there is no prior matrix that can be used, the model assumes a matrix where each trip is equally likely. This means that the matrix selected by the program will be the most evenly distributed matrix that reproduces the observed traffic counts well.

Setting the new matrix that was estimated using ME2 as the prior matrix, ME2 is run again. An OD matrix is found that reproduces the input traffic counts, and resembles the new prior matrix. This process is repeated again and again, iteratively refining the estimated trip matrix until there is little change in the OD matrix predicted between iterations.

For a more detailed explanation of the theory underlying the ME2 matrix estimation procedure the reader is referred to Van Zuylen and Willumsen (1980).

The method described above assumes that it is possible to obtain the p_{ija} factors independently of the OD matrix estimation process. In congested networks however, the two will be interdependent. This is because the p_{ija} factors are dependent upon the traffic flow on each link, as the flow on the link will affect travel times, and hence the cost and desirability of a route. Similarly, the origin-destination matrix that is estimated will also be dependent upon the traffic flow counts on each route, as ME2 uses these to select the OD matrix. Hence there is interdependence between the two.

To overcome this problem with interdependence, SATURN uses an iterative approach, initially assuming a set of p_{ija} factors. An OD matrix is then estimated and loaded onto the network to obtain a new set of p_{ija} factors. This process is repeated until the p_{ija} factors and estimated OD matrix are mutually consistent.

7.3.3 Simulating the Current Travel Regime

If a driver is travelling from an origin A to a destination B, the driver will select a route through the network that minimises his or her perceived cost of the journey. This cost of the journey will be a function of the travel time, the trip distance, the perceived level of safety along each route, its reliability, and the drivers' personal tastes.

Wardrop theorised in 1952 that drivers will act in a “selfish” manner, selecting a path through the network that will minimise their own personal cost of travel. This theory, known as Wardrop’s equilibrium, was formalised as:

“Under equilibrium conditions traffic arranges itself in congested networks in such a way that no individual trip makers can reduce his (or her) path costs by switching routes.”

Under these equilibrium conditions, the user-optimal flow pattern may be found mathematically by minimising the following equation:

$$\text{minimise } z = \sum_a \int_0^{f_a} C_a(x) dx \quad (7-3)$$

where $C_a(x)$ is the travel cost on link a when the flow rate is equal to x , and the flow rate on link a is f_a .

The true cost of any journey will be a complex function of many parameters. For modelling purposes, the cost is simplified to only a function of the trip travel time and the distance travelled. The distance along a particular route is easily quantified, but the travel time along that route will be dependent upon the number of vehicles using the road. As vehicle congestion on a road increases, the average travel speed will decrease, and the travel time and hence the travel cost will increase.

To illustrate how Wardrop’s principle works, lets take an example where there are two possible routes for travellers to use, link a or link b (Figure 7.1).

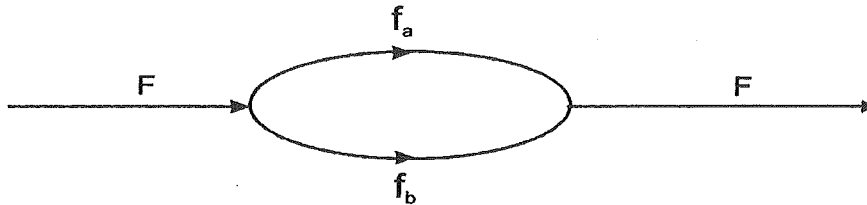


Figure 7-1: Example network, with two possible flow paths, a and b

The cost of travelling along either of these links will be dependent upon the flow on each of the links, as shown in Figure 7-2.



Figure 7-2: Cost on each link as a function of the number of vehicles using the road

For Wardrop’s equilibrium to be satisfied, the following objective function must be minimised:

$$z = \int_0^{f_a} C_a(x)dx + \int_0^{f_b} C_b(x)dx = z_a + z_b \quad (7-4)$$

Figure 7-3 shows that this function is minimised when $C_a(f_a) = C_b(f_b)$, and when $f_a + f_b = F$. Movement to either the left or the right of the point where these two cost functions are equal, results in an increase of ΔZ under the cost curve, and hence a greater cost of travel for any one traveller (Figure 7-4). This implies that some travellers would benefit from changing routes. Wardrop's equilibrium only stands when no traveller would benefit from switching routes.

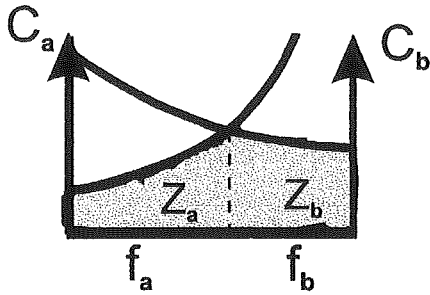


Figure 7-3: Flows on each link in equilibrium, so that costs are minimised

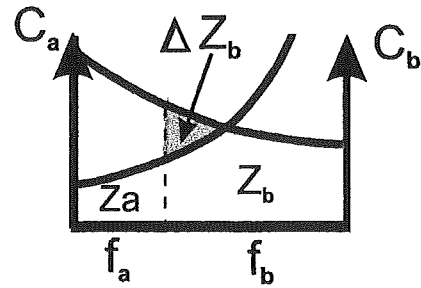


Figure 7-4: Costs of travel not minimised

When SATURN actually assigns trips onto a network, it works towards an arrangement that satisfies Wardrop's equilibrium iteratively. Initially, the program assigns flow to certain links in an all-or-nothing type assignment. This is where all of the trips from one origin to one destination choose the same route with no account taken of the traffic flow on that route. With the trips loaded onto the network, the link travel times are calculated with the current traffic density. New costs of travel on each link are calculated given these travel times. Using these new costs of travel, an auxiliary, all-or-nothing assignment of trips is made. The improved set of trip assignments is then found as a linear combination of the old and auxiliary flows using Equation 7-5.

$$V_a(n+1) = (1-X) * V_a(n) + X * F_a(n) \quad (7-5)$$

where $V_a(n)$ is the assigned flow on link a after the nth iteration, $F_a(n)$ is the auxiliary flow on link a after the nth iteration, and X is chosen so that the new flow $V_a(n+1)$ minimises the total cost of travel for all users on the network.

Again, the costs of travelling on each link for the current level of congestion are calculated, and another auxiliary, all-or-nothing assignment is made. This process is repeated until successive assignments produce little change in the flow rates on the links.

Under congested conditions, this user equilibrium (UE) approach will produce good predictions of traffic behaviour, with vehicles distributed throughout the network. When there are low flows on the network however, congestion will not be inhibiting travel, and the cost function will be nearly independent of the flow on each link. Because of this, the model will predict that all vehicles travelling from point A to B

will use the same route, as this route has the minimum cost associated with it. This results in nearly all of the traffic travelling on only a few links in the network, whilst others have no flow at all. This obviously does not occur in reality, as not every driver with the same origin and destination will take the same route. The differences in route selection may be because the driver perceives that the cost on one route is less than on the other. One driver's perception of the cheapest route may not coincide with another's.

To account for this difference between the perception of drivers, a stochastic user equilibrium (SUE) model is used. This model assumes that the cost of travelling on a route has a probability distribution, varying about the calculated cost of travel. This effectively simulates the variation that would occur from the differing perceptions of drivers about the desirability of routes.

To assign trips onto the various routes using SUE, SATURN initially randomly generates costs of travel along each route from their respective distributions. It is assumed that perceived costs are uniformly distributed, varying between C_1 and C_2 , and about the mean (calculated) cost $\left(C = \frac{(C_1 + C_2)}{2} \right)$, with the probability function:

$$f(c) = \frac{1}{C_2 - C_1} \quad (7-6)$$

where $(C_2 - C) = (C - C_1) = 0.2 * C$. The value of 0.2 is a default value suggested by the developers of the SATURN program. In theory, it would be desirable to use a normal distribution to describe the variation in perceived cost of using each link. SATURN does permit a normal probability density function to be used, but the increased computer time required for convergence to be achieved with a normal distribution outweighs the increased realism attained (Van Vliet, 1995).

The cost of travel on each link, generated randomly from their respective distributions, is used in an all-or-nothing assignment of trips onto the network. New costs of travel on each link are calculated, accounting for any congestion there may be in the network under the assigned flow pattern. The perceived costs of travel on each link are then again generated randomly from a uniform distribution varying about this calculated cost (C). Similar to the user equilibrium (UE) assignment, a set of auxiliary all-or-nothing flows are then assigned using the newly found cost of travel when the links are congested. The assigned and auxiliary flows are then combined using Equation 7-5. This method is repeated until successive iterations produce little change in the assigned flows.

7.3.4 Confirmation of Model Reality

A traffic model, such as that developed here, can only ever be just that -a model of the network that exists in reality. For this reason there will always be limitations to the accuracy with which a model can predict traffic behaviour. When developing a model, the aim of the developer is not therefore to develop a model that is perfectly consistent with reality, as this is usually unobtainable. The aim should be to ensure

that the model has a much realism as possible in the areas that are going to be of most importance. For this study, it is of most benefit to the project if the model can make reasonable predictions of traffic behaviour in the Central Plateau region, particularly the Desert Road and its alternative routes.

The best way to assess how well the traffic model is predicting traffic behaviour, is to compare traffic flows predicted by the model, to the observed traffic flows under normal conditions. Looking at how well the model predicts traffic flows on the major routes of interest, it can be seen from Table 7-2 that the model is performing very well. It is predicting hourly traffic flows to within 3% of those observed, which is a very good prediction considering the size of the network modelled.

Table 7-2: Comparison of observed and predicted flows for the Desert Road and its alternative routes

State Highway N ^o	Between Nodes... ¹	Observed Flows (veh/hr in both directions)	Predicted Flows (veh/hr in both directions)	% Difference
1	13 & 26	150	154.1	2.7%
4	15 & 24	100	102.9	2.9%
47	14 & 56	64	65.7	2.7%
49	25 & 26	70	70.1	0.1%

When looking at the performance of the overall network, it is necessary to compare the accuracy of predictions on links that have vastly different levels of flows. A difference between the observed and predicted flows of 100 vehicles per hour would be considered a large difference if the flows are of the order of 100 vehicles per hour. A difference of 100 vehicles though fades to insignificance if the total flows are several thousand vehicles per hour. Similarly, a 10% difference is unimportant if the observed flow was only 10 vehicles per hour, as this is only a variation of 1 vehicle per hour. Conversely, if the flow is several thousand, a 10% difference in vehicle flows is significant. To overcome these problems in flow comparison, a GEH statistic is used. The GEH statistic is the square root of the product of the absolute difference between observed and predicted flows, and the relative difference (Equation 7-7):

$$GEH = \sqrt{\frac{(V_2 - V_1)^2}{\frac{1}{2}(V_1 + V_2)}} \quad (7-7)$$

where V_2 is the larger of the flow predicted by the model and the observed flow, and V_1 is the lesser of the two.

The SATURN manual (Van Vliet, 1995) suggests that as a rule of thumb, a GEH statistic of 5 or less indicates an acceptable fit. Links with a GEH statistic greater than 10 would probably require closer attention. Of the 170 links modelled in the

¹ See map in Appendix 7 for the location of nodes.

Central North Island network, 89 percent of them have a GEH statistic of two or less; 7% between 2 and 4; 3% between 4 and 6; and the final 1% between 6 and 8. These statistics are evidence that the traffic model is performing very well. It is predicting with good accuracy, the traffic flows and behaviour over the whole of the network.

7.4 Costs of Traffic Movement

The costs of travel on the road network have been defined in accordance with the recommendations laid out in the current Project Evaluation Manual (PEM) published by Transfund New Zealand (1997).

The costs of making a particular trip will include vehicle running costs, such as petrol consumption, tyre and brake wear, and general maintenance required to keep a car running in good order. These in turn will be a function of the characteristics of the road upon which the trip is made. Factors such as brake wear and petrol consumption will be dependent upon the road alignment and grade, whilst tyre wear and general maintenance requirements will be dependent on other factors such as driving speeds, road roughness, and traffic delay cycles. There is also a time cost of travel, where vehicle occupants lose the opportunity to spend travel time in some other activity. The cost of this travel time will be dependent upon the type of trip that is being made (e.g. work related trips or a Sunday drive), the characteristics of the occupants, and the degree of comfort the occupants are experiencing during the trip.

For simplification in traffic modelling, the cost of making a trip is defined as a function of only two variables; the distance that is travelled, and the time taken to cover that distance. The distance travelled will be defined by the route taken to get from the origin to the destination. The time taken to travel along that route will be a function of the route congestion. Travel speeds on a road link are dictated by the number of cars using that particular road, in relation to the road's capacity.

7.4.1 Vehicle Operating Costs

The PEM sets out standardised costs for each kilometre of travel, that can be used for the purposes of benefit-cost analysis. These costs are assigned according to the vehicle class, and are also provided for standard traffic compositions. Standard traffic compositions differ according to the type of road, with four road categories defined in the PEM. Most of the roads included in the Central North Island road network are State Highways, and constitute arterial links between major centers. These characteristics fall into the "rural strategic" category specified in the PEM. There are some small road links (Average Annual Daily Traffic (AADT) < 1500) included in the model, but these constitute only 7% of the observed vehicle kilometres travelled on the network. The larger road links are the most important in terms of analysis and costing, so the assumption that all roads modelled in the network fall within the "rural strategic" category is appropriate.

The costs of fuel, tyres and tubes, repairs and maintenance, oil, and the proportion of depreciation related to vehicle use, are included in the vehicle operating costs defined in the PEM. They do not include costs that would be incurred regardless of whether or not the vehicle is used. The operating costs are tabulated for each road category in terms of the speed of travel and the gradient of the road. To find the total cost of road travel, it is necessary to determine an appropriate speed and gradient that can be used to find the average vehicle operating costs over the whole of the network.

When simulating travel through the modelled network, it is possible to predict what the average speeds on each link will be, and the average speed of traffic over the whole network. These predictions account for the effect of traffic density on the speed of travel. The average speed predicted for the whole network under normal conditions (i.e. with no road closures) is 86 km/h.

Determining an appropriate gradient to use is more complex. Tables in the PEM specify travel costs depending upon the gradient of the road section, ranging between uphill and downhill gradients of 10%. Such detailed analysis of each section of road is not feasible in this study. Only an average vehicle operation cost for travel over the whole of the Central North Island network is required. As the adage says, what goes up must come down, and so it would be expected that the average gradient over the whole network would be 0%. The cost of a vehicle travelling up a hill and then down the other side, will be more than the same distance travelled on level ground, even though the average gradient is the same. For this reason, it would be expected that the average cost of vehicles travelling on the network, would be more than if the whole of the network were level. For the purposes of this study, an assumed gradient of +1.5% was used to determine the appropriate unit cost of travel per kilometre from the PEM.

Using these speed and gradient values, the running cost in cents per kilometre for typical vehicles on a rural strategic road is 32.4 cents/km (Table A5.15a, Transfund New Zealand, 1997).

Other characteristics that the PEM factors into the cost of vehicle operation are the roughness of the road surface, speed cycle changes, and vehicles idling while traffic flow is stopped. It is not anticipated that closing of any of the roads will alter any of these factors. It is therefore possible to exclude them from the analysis.

The vehicle operation costs defined in the PEM are expressed in terms of July 1994 dollar values. These need to be factored into July 1997 dollars (the base year for project costs) prior to use in the final benefit-cost analysis. This amounts to a vehicle operation cost of 34.1 cents/km in July 1997 dollars.

7.4.2 Occupant Travel Time Costs

Time spent by vehicle occupants travelling from their origin to their destination has an opportunity cost as that time, if not spent travelling, could be used for other purposes. Because of this lost opportunity, it is necessary to place a value on the time spent by travellers en-route to their destinations. Savings in travel time are a

tangible benefit to society in terms of an increased availability of human resources. The potential productivity of this resource, and the benefits of travel time savings, will be dependent upon the characteristics of the vehicle occupant. People value their time differently depending upon the purpose of a trip. Trips that are taken for the pure enjoyment of the travelling experience (e.g. the Sunday driver) will have a very low value of travel time. A trip taken when time is at a premium, will have a very high value of time, and hence there will be more benefit from travel time savings. Similarly, the degree of comfort that a traveller is experiencing will influence the value they will place on any potential time savings. For example, a person standing on a bus will value time savings more than a car passenger who is comfortable with their surroundings. This implies that the time cost of travel is higher for the standing bus passenger.

The PEM recognises this spectrum of travel time valuations, providing costs of travel time that can be used for various occupant types, depending on whether or not they are travelling during work-hours. The PEM also provides estimates for composite values of travel time for typical travellers on roads falling into defined categories. Once again, the “rural strategic” category was used to describe occupant characteristics of vehicles throughout the modelled network. Table A4.2 of the PEM specifies travel time costs to be used in dollars per hour for vehicles on rural strategic roads, divided into two time periods. For travel on a weekday, a cost of \$22.20/hr is specified, whereas on weekends or holidays the cost to be used is \$16.00/hr. Evaluations covering all time periods use a composite value of \$20.10/hr.

The value used in this model was the composite value of \$20.10/hr. The model has not been designed to differentiate between closures that occur during week days, or those over the weekend. The model assumes similar traffic flows on all days of the week, and that hazards are no more likely to occur on any one day than another. Composite values are therefore appropriate.

As with the vehicle operation costs, the above values are in July 1994 dollars, equating to an occupant travel time cost of \$21.30/hr, or 35.5 cents/minute in July 1997 dollars.

7.4.3 Cost of Traffic Accidents

Before discussing how the cost of traffic accidents on the road network are calculated, it is worth mentioning that the cost of accidents is assumed not to affect which route drivers choose to use. The SATURN model assumes that only the vehicle occupant’s time and on-road vehicle costs are considered. When calculating the total cost of road closures to the New Zealand economy though, Transfund New Zealand does require that the increased accident cost of longer trip distances be taken into account. It is for this purpose that the method of calculating accident costs is detailed.

Appendix 6 in the Project Evaluation Manual (Transfund New Zealand, 1997) outlines the approach required for the analysis of accident cost in the evaluation of projects. Closures of the Desert Road, or any of its alternative routes, will cause

traffic to divert onto other roads in the road network. These diversions will usually increase the total distance of travel required for the driver, and also his or her exposure to accidents on that trip. This effect must be accounted for when assessing the overall cost of road closures.

The PEM states that an analysis of the change in accident cost must be evaluated individually for each link in the road network, as the accident cost will be a function of more than just the total amount of travel on the network:

“When projects of an area-wide nature are being considered, such as the evaluation of an urban traffic network, it is insufficient to calculate accident costs from changes in global totals of vehicle-kilometres of travel.

“Where a new road link is being added to a network, efforts should be made to analyse the incidence of accidents on those links on which most of the diversion effects will be experienced.”

For mid-block sections of road, typical injury accident rates are determined by:

$$Injury\ accidents\ per\ year = b * X \tag{7-8}$$

where X is the exposure in 100 million vehicle kilometres, and the coefficient b is a factor selected from Table 7-3.

Table 7-3: Values for coefficient "b" for 100km/h speed limit two-lane roads (Transfund New Zealand, 1997)

AADT	Terrain Type		
	Flat	Rolling	Mountainous
< 2,500	25	28	40
2,500 - 12,000	18	25	38
> 12,000	13	23	n/a

Both the number of people injured in an accident, and the severity of any injuries, will generally increase with the speed at which the vehicles involved were travelling at the time of the accident. Any change to travellers' behaviour during a road closure that causes their mean travelling speed to change, will therefore also affect the severity of accidents that might result. Similarly, increased congestion on alternative routes, and subsequent slowing of traffic on those roads, will also alter the characteristics and therefore the cost of accidents along that road link.

The PEM recommends that where there is a change in mean speed expected, the cost of accidents should be adjusted to reflect this change using Equation 7-9 (for speeds ranging from 50 to 100km/h).

$$Adjusted\ accident\ costs = accident\ cost * \frac{V_2 - 30}{V_1 - 30} \tag{7-9}$$

where V_1 is the mean speed for the existing conditions, and V_2 is the mean speed that could be expected under the altered conditions.

As the network of roads that has been studied here is extensive, some simplification of the analysis of accident costs is necessary. There are two main factors that will change the cost of accidents on the network when a road link is closed. Firstly, there will be a reduction in the number of trips made, as the increased cost of trip making deters some travellers. The reduction in the number of trips made will decrease the total number of vehicle kilometres travelled. The second change will come about as travellers that would normally use the route that is closed, have to divert onto other roads to get to their destination.

Of the roads that are included in the modelled network, most fall into the “rolling” category in Table 7-3. There will be some exceptions, where particular road links are at least partially more flat or mountainous than the rest, but the roads of most interest (i.e. the Desert Road, and its alternative routes) do fall into the “rolling” terrain category. As estimates of the change in accident cost are not so sensitive to the classification of links where there is no significant diversion of traffic, the assumption that all road links in the network are “rolling” is a reasonable simplification.

It can be seen from Table 7-3 that the accident rate coefficient b for the “rolling” category is 28 for road links with an AADT less than 2,500, and 25 for links with AADT flows between 2,500 and 12,000. Using these coefficients as defined, would result in a large change in accident costs being predicted for changes in flows that moved a road link from one AADT category to another. Another road link, with a similar change in flow rate, that did not change AADT category, would not show such a marked change. To reduce this effect, and to reflect the gradual change that could be expected in accident rate with AADT, the linear relationship given in Equation 7-10 was fitted to the above categories (Figure 7-5).

$$b = -0.0005 * AADT + 28.625 \quad (7-10)$$

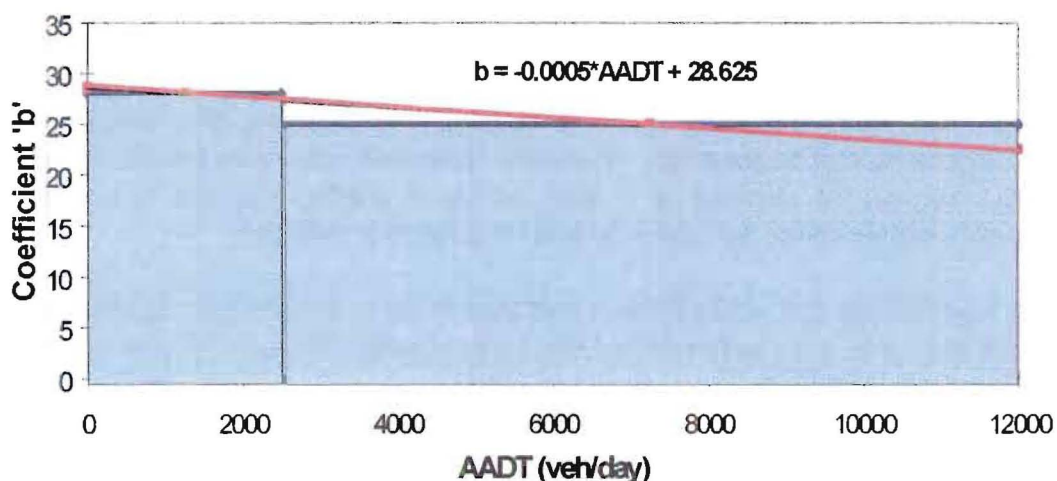


Figure 7-5: Linear relationship fitted to AADT categories for the accident rate coefficient b

In this study, the only factor altering travel speeds on a road link will be increased congestion when some roads are closed. This study does not investigate any projects that are likely to significantly alter the travel speeds on the road link themselves. For this reason, to use Equation 7-9 to adjust accident costs for changing travel speeds, would be double counting the effects of congestion on the roads. The impact of increased congestion on the roads will already be accounted for in the accident exposure rate used to calculate the number of accidents that can be expected, as these are dependent upon the AADT on the links.

Classifying all roads by their average annual daily traffic (AADT), the average annual number of injury accidents is then found using Equation 7-8 and the accident coefficient b from Equation 7-10. The cost per reported injury accident in July 1994 dollars is \$514,000 (figure suggested for use in the PEM). To adjust these costs into July 1997 dollars, a factor of 1.06 is used.

This method will provide an estimate of the annual cost of accidents on the road network when all road links are open. To find the change in accident cost when the Desert Road is closed, each link must then be individually assessed. The difference between the total cost of accidents when the Desert Road is closed and the total cost of accidents when all road links are open, will be the cost of accidents caused by closure of the Desert Road.

An alternative approach could be to use statistics from the Land Transport Safety Authority (LTSA) to find the average annual number of reported accidents for each individual road link on the road network. There will usually be some under-reporting of accidents, particularly of accidents that do not result in injuries. To find the total number of accidents that occur on a section of road, the number of observed accidents would need to be factored to account for this under-reporting. The factors suggested by the PEM to calculate the total number of accidents from the number of reported accidents are:

- fatal = 1.0;
- serious = 2.0;
- minor = 4.0;
- non-injury = 35.

The ratio for serious accidents seems to be high considering that it is likely that emergency services would be called to an accident causing serious injuries. These values from the PEM have been used in this model, but it may be worth checking the validity these ratios as part of any further work.

To cost these accidents, they may then be classified by the severity of each individual accident, the movement that led to each accident, and the type of vehicles involved. Assessing each accident to this level of detail though, would not be appropriate. The Desert Road, being part of the SH1, has a relatively high proportion of line-haul vehicles using it. During a closure these vehicles would be diverted onto the alternative routes, altering their traffic composition and possibly altering the type of accidents that may be expected. The costing should be simplified to only being a function of accident severity, in an effort to reduce the amount of implied accuracy in the accident cost estimates. Thus the cost of each accident,

regardless of the type of vehicles involved or the movements leading to the accident are:

- cost per fatal accident = \$2,570,000;
- cost per serious accident = \$236,000;
- cost per minor accident = \$21,400;
- cost per non-injury accident = \$2,400.

These costs are all in July 1994 dollars. To adjust them into July 1997 dollars, a factor of 1.06 was used. These costs are valued to reflect the monetary value of life (for fatalities), pain, grief, and suffering, the costs of hospital care, and any loss of productivity.

If further refinement of the accident costs is required, then this more complex method, working with observed accident rates, could be further investigated.

7.5 Modelling Traffic Behaviour During Road Closures

To model the closure of a particular road link, that road link is removed from the buffer traffic network that is input to SATURN. With this link no longer an option for travel, SATURN proceeds in its normal way, routing travel through the network on the paths of least perceived cost.

SATURN predicts traffic flows on an hourly basis, so the cost per hour of road closure can be estimated. To find the total cost of a closure, this hourly rate is then multiplied by the total duration of the road closure. One of the assumptions SATURN uses is that traffic flow is in equilibrium. Just after a road has been closed however this equilibrium condition will not be present. How long it takes for equilibrium flows to be reached after the road has been closed is uncertain. At the time that the road is being closed, there will be some trip makers already in the network who did not know that the road was going to be closed. SATURN assumes that travellers have a perfect knowledge of the road network and its characteristics (and would therefore know if a road is closed, and what are the best alternative routes) and will always choose the path of least cost. If a link is closed part way through a traveller's journey though, they may have already passed the junction to the next best alternative route, and will therefore either have to back-track to get to this route, or select the next best alternative. Thus the cost of road closure for this traveller will be greater than that predicted by the model. This effect would be most noticeable for short closures, where the inflated cost of diversion will be proportionately large compared to the total cost of closure. Countering this though, is that if the closure is only expected to be short, drivers may simply take a rest stop, and wait until the preferred route is reopened. If the travelling public is well informed about the status of links and know which is the best alternative route, then the cost of traffic diversions may be minimised. Variable message signs are an example of a way to update travellers with this timely information.

Work is currently being done at the Department of Civil Engineering, University of Canterbury to establish how much error is associated with using SATURN to assess

conditions where equilibrium has not been reached. The results of this new research should be assessed when they come available to evaluate the validity of this model using SATURN predictions.

An extension to the traffic model that may be worthwhile, is the differentiation between trips made during certain time periods. As can be witnessed during rush hours, the density of traffic flow does not remain constant over a 24 hour period, as the model developed here assumes. Instead, it has periods of peak flow followed by quieter periods. Although not so marked on a rural road, there are still flow fluctuations over a 24 hour period. In particular, during the day time (from 8am to 6pm), nearly 75% of the total daily trips will be made. This means that hazards with a tendency to occur during this period will be disruptive to a greater number of trips than a hazard that usually occurs during the night. The disruption to a greater number of trips will increase the relative cost of these closures. The greater impact of day-time closures may become significant when comparing a hazard such as ice formation, which typically occurs during the night, with a hazard such as earthquakes that have no such time dependence.

7.5.1 Elasticity of Travel

The elasticity of driver behaviour describes the willingness of drivers to alter their travel plans, based on what the trip will cost (Equation 7-11). For example, if one link in the network is closed and traffic has to divert, the price of the trip will increase. Some travellers may find that this new price is too expensive, and will cancel their trip. The percentage of travellers that cancel their trip, divided by the percent change in the trip price, will be the elasticity exhibited by the users of that road link.

$$\text{Elasticity} = \frac{(\% \text{ change in number of trips made})}{(\% \text{ change in the price of a trip})} \quad (7-11)$$

When a road link is closed in the network, it will cause more disruption to some travellers than others. Trips from some origins to some destinations will have readily available alternative routes, whilst other will require significantly longer distances to travel. This means that for a given elasticity, more trips will be cancelled for trips that have a greater increase in cost, where more disruption to the trip is caused.

SATURN has a built in feature that enables it to predict traffic behaviour under elastic conditions. The way it does this is to effectively build a pseudo network, that links together each origin and each destination. This pseudo link is what the program assumes that all trips that are cancelled “use”. The cost of travel on this pseudo link is priced according to the elasticity of the travellers, so that during assignment of flows onto the network, a proportion of the trips will use this pseudo link. Vehicles on this link are those trips that are assumed to cancel their trip due to the increased cost of travel.

To represent the willingness of people to still travel when the price of the trip has increased (i.e. the elasticity of the travellers), it is assumed that the number of trips still made will follow a power relationship (Equation 7-12).

$$T_{ij} = T_{0ij} \left(\frac{C_{ij}}{C_{0ij}} \right)^{-P} \quad (7-12)$$

where T_{ij} is the number of trips that will be made from i to j when the trip costs C_{ij} , T_{0ij} is the number of trips that were made when the cost of the trip was only C_{0ij} , and P is the parameter of elasticity.

Traffic flows that were observed during the nine day closure of the Desert Road in 1995 were used to find how much elasticity drivers on the Central North Island network exhibit. Two continuous traffic counting stations, one on State Highway 1 just south of Waiouru, and the other on State Highway 1 to the north of Turangi (i.e. on State Highway 1 just to the north and south of the Desert Road), both showed that there was a reduction in flow on these links of around 1600 vehicles per day during the closure (Works Consultancy Services, 1996). Several values of P were trialed, and the reductions in flow predicted when the Desert Road was closed were compared with the observed reductions in flow. It was found that when P was set to equal 3.0, that the model predictions best approximated the observed traffic behaviour.

It should be noted that the ability to model elastic traffic behaviour is relatively new, and that there have been few studies investigating the degree of elasticity expressed by particular road user groups. A result of this is that there is very little guidance available as to what constitutes a reasonable elasticity parameter, or how stable values of elasticity for a group of travellers are (i.e. does the expressed elasticity fluctuate with time or other circumstances). The estimate of the elasticity of travellers on the network used in this study was found from observations of traffic behaviour when the Desert Road is closed. It shall be assumed that this elasticity is also representative of the behaviour that would be expressed during other road closures, or concurrent closures of a number of links.

Estimation of driver elasticity is an area that is likely to benefit from further study. In particular, there will probably be different elasticities expressed by travellers during short and long closures, and during closures with different causes. At present the magnitude of these differences is unknown.

Those travellers that choose to cancel their trip will lose the benefits that would have been gained from making the trip. The traffic network model provides a means to estimate the value of this lost benefit. When all road links on the network are open, there are a certain number of trips that will be made from origin i to destination j . This means that the implied benefit of the trip from i to j must be greater than the cost of making this trip, or else they would not have made the journey. The cost of making that journey will increase when a road link is closed. If travellers then decide to cancel their trip, then this implies that the benefit of making the trip is less than this new cost of travel. The value of the user benefit from making the trip is

assumed to be half way between the original cost of travel, and the new cost of travel for which the trip is cancelled (Neuburger, 1971). That is:

$$UB_{ij} = \frac{1}{2}(C_{ij} + C_{0ij}) \quad (7-13)$$

where UB_{ij} is the user benefit of making one trip from i to j , and C_{0ij} and C_{ij} are the costs of travel when the trip is taken and when it is cancelled.

7.6 Economic Cost of Road Closures

Closure of any of the road links in a network will result in a net cost to the New Zealand economy. Any travellers that would normally use that link will be forced to either divert or cancel their trips. Those that divert will incur additional vehicle operation costs, vehicle occupant travel time costs, and costs due to the increased exposure to accidents. Those that cancel their trip will lose their user benefit of that trip.

The economic analysis done in this study adheres to the general philosophy behind the PEM (Transfund New Zealand, 1997). The PEM looks at the costs and benefits from a national viewpoint, where the net costs and benefits of a project are those to the whole of the New Zealand economy. It should be noted that this does not account for the distributional effects of a project, where a project may benefit one region or section of the population, while disadvantaging another. If a project reduces benefit to one section of the community but increases benefit to another, then the resulting net benefit to the whole of the New Zealand economy may be zero. Benefits are neither created nor reduced; they are simply transferred (Copeland, 1979).

During recent closures of the Desert Road, there have been many complaints by businesses in Waiouru that they are losing custom due to reduced traffic flows through their community. The economic impact of this lost business is not included in any analysis however, as traffic diversions away from these businesses will presumably be taking travellers past other business in the network and increasing their profitability. From a national viewpoint, this increased profitability will balance the losses incurred at Waiouru. Even if there is a net difference in profitability however, this effect would still not be incorporated. The reasoning for this was given in Prest and Turvey (1965) as follows:

“[An example of where external cost and benefits should not be accounted for is...] when the improvement of a road leads to greater profitability of the garages and restaurant on that road, employment of more labour by them, higher rent payments to the relevant landlords, etc. In general this will not be an additional benefit to be credited to the road investment, even if the extra profitability, etc, of the garages on one road is not offset by lower profitability

of garages on the other roads, which are now less used as a result of the traffic diversion. Any net difference in profitability and any net rise in rents and land values is simply a reflection of the benefits of more journeys being undertaken etc. than before and it would be double counting if these were included too.”

There may be some use however in identifying projects that are likely to have substantial distributional effects and those which will not. This information, although not included in the actual benefit-cost ratio for the projects, could then be taken into account by those making any decision. This study has not attempted to identify or quantify distributional effects that could be attributed to road closures, primarily due to the complexity of issues that would need to be considered. There will also be other less tangible social consequences of road closure, such as community isolation, that have not been considered in this study. The area of social cost of road closures is one that could benefit from further investigation at a later stage.

7.7 Comparison of Risk Imposed by the Hazards

As was discussed earlier, the cost of road closures is a function of the costs of making the trip (both vehicle operating costs and the costs of vehicle occupant time), the cost of accidents that may occur during the trip, and lost user benefit from those trips that are suppressed. The cost per hour of each of these factors, as well as the overall hourly cost of the road closure, is given in Table 7-4 for the major closure scenarios that were considered in this study.

Table 7-4 shows that closure of the Desert Road alone costs the New Zealand economy nearly \$8,000 per hour of closure. When the other major alternative North-South road link (SH 4) is also closed, detour lengths are significantly increased, and the cost of road closure is inflated to nearly \$23,000 per hour. In contrast, simultaneous closure of other State Highways near to the Desert Road, that do not pose a significant barrier to the North-South traffic (such as SH 47 and SH 49) are not so costly. This indicates that the impacts of road closures are highly dependent upon the characteristics of the road network and availability of alternative routes for travel. The most costly closure scenario is when the Desert Road, SH 4, SH 41, SH 47 and SH 49 are simultaneously closed.

Figure 7.6 compares the cumulative frequencies of the annual cost of closures that can be expected from each of the hazards that were studied. This shows that snow and ice is the most significant hazard, costing the New Zealand economy an average of 1.86 million dollars a year.

Table 7-4: Costs per hour of various road closure scenarios

Road Closure Scenario	Cost of Making the Trips (\$/hr)	Accident Costs (\$/hr)	Cost of Lost User Benefit (\$/hr)	Total Cost (\$/hr)	Total Cost of Closure (\$/hr)
All Roads Open	180,590	43,017	N/A	223,607	N/A
Desert Road Closed	172,610	40,858	18,129	231,597	7,990
D Rd ² and SH 1 ³ Closed	169,646	39,679	26,855	236,180	12,570
D Rd and SH 4 ⁴ Closed	164,412	38,570	43,498	246,480	22,870
D Rd and SH 41 ⁵ Closed	171,682	40,676	20,491	232,849	9,240
D Rd and SH 47 Closed	165,220	38,846	33,887	237,953	14,350
D Rd and SH 49 Closed	169,944	39,783	26,058	235,785	12,180
D Rd, SH 1, and SH 49 Closed	168,615	39,457	26,768	234,840	11,230
D Rd, SH 4, and SH 47 Closed	161,336	37,839	47,117	246,292	22,690
D Rd, SH 4 and SH 49 Closed	164,064	38,459	44,703	247,226	23,620
D Rd, SH 4, and SH 41 ⁵ Closed	163,551	38,428	45,736	247,715	24,110
D Rd, SH 41 ⁵ , and SH 47 Closed	161,329	38,061	44,743	244,133	20,530
D Rd, SH 41 ⁵ , and SH 49 Closed	169,025	39,607	28,402	237,034	13,430
D Rd, SH 47 and SH 49 Closed	163,311	38,088	38,665	240,064	16,460
D Rd, SH 4, SH 47 and SH 49 Closed	160,992	37,728	48,316	247,036	23,430
D Rd, SH 4, SH 41 ⁵ and SH 47 Closed	157,766	37,098	56,715	251,579	27,970
D Rd, SH 4, SH 41 ⁵ and SH 49 Closed	163,202	38,310	46,942	248,454	24,850
D Rd, SH 41 ⁵ , SH 47 and SH 49 Closed	159,497	37,316	50,434	247,247	23,640
D Rd, SH 4, SH 41 ⁶ , SH 47, and SH 49 Closed	156,523	36,818	61,322	254,663	31,060
D Rd, SH 4, SH 41 ⁵ , SH 47 and SH 49 Closed	157,228	36,938	59,388	253,554	29,950
D Rd, SH 1, SH 4, and SH 49 Closed	162,713	38,130	45,397	246,240	22,630
D Rd, SH 1, SH 4, SH 47, and SH 49 Closed	159,640	37,400	47,652	244,692	21,090
D Rd, SH 1, SH 4, SH 41 ⁵ , SH 47, and SH 49 Closed	155,473	36,557	59,547	251,577	27,970

² D Rd stands for the Desert Road.

³ Taihape to Waiouru section of SH 1.

⁴ Raetihi to National Park section of SH 4.

⁵ Turangi to the SH 32 section of SH 41

⁶ Manunui to Turangi section of SH 41.

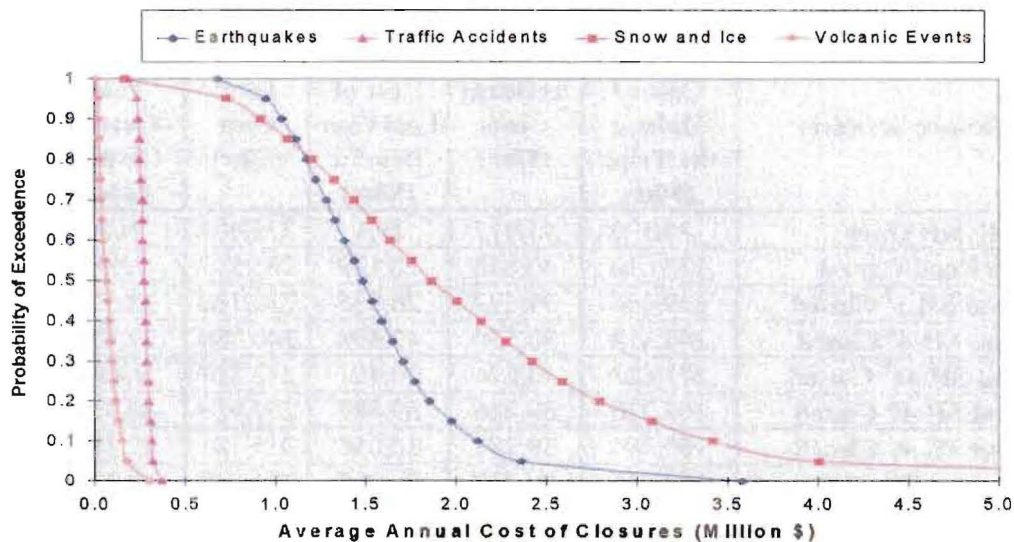


Figure 7-6: Comparison of the annual cost of closures due to each closure mechanism

There is a small chance that closure costs due to earthquakes will exceed those due to snow and ice. To confirm which hazard is dominant, the area under the cumulative frequency distributions are compared. The area encompassed between the two cumulative distributions, below the intersection of the two curves, is much greater than the area above the intersection. This means that snow and ice exerts second-degree stochastic dominance over the risk caused by earthquakes (Tai, 1987), confirming that snow and ice is the most significant hazard.

It should be noted that the cost of closures due to earthquakes shown in Figure 7-6 assumes that the road will not re-open until bridge repairs have been completed. In reality, it is likely that some sort of temporary access will be put in place to resume traffic flow. This means that the annual cost of closure due to earthquakes shown in Figure 7.6 exaggerates the risk posed.

Traffic accidents and volcanic events pose much less threat to the Desert Road than do snow and ice or earthquakes. Of the risk caused by volcanic events, the majority of this risk is due to lahar hazard, with little annual risk caused by ashfall. It should be noted that the risk calculated for volcanic events, only looks at the impacts of ashfall on the roads within the Central Plateau region. Wider geographic effects, over a much larger proportion of the New Zealand may be experienced following a large eruption. This would significantly increase the cost of disruption caused by an eruption, and may be an area worth further investigation.

The slope of the cumulative distributions indicate the level of uncertainty there is in the predictions of the annual cost of closures. It can be seen from Figure 7.6 that there is a relatively large spread in the annual cost of closures predicted for snow and ice. This large spread is indicative of the variability of the observed closure data, and hence the closure durations and probabilities predicted by the model. It is interesting to note that although there is a large spread in the predicted annual costs, snow and ice is still the distinct major closure hazard. This is especially true when

looking the 95th percentile predictions, with very high annual costs (\$4 million) possible.

7.8 Conclusions

The SATURN model of the traffic network for Central North Island appears to predict the behaviour of traffic under normal conditions, when all road links are available, very well. This provides confidence that it will also be of use in predicting traffic behaviour when one or more of the road links are closed. It should be noted though that the traffic model is just that; a model of what actually occurs in reality. There will always be simplifications necessary, and uncertainties about the traffic characteristics modelled. Overall though, the model results can be viewed with a reasonable level of confidence. The traffic model provides a substantial starting base, from which further enhancement can evolve.

Comparison of the predicted annual cost of closures from each of the hazards indicates that snow and ice is the most significant closure mechanism, costing the country on average 1.4 million dollars annually. There is variability in the annual cost predicted by the model for snow and ice, reflecting the variability in the observed closure data. Even with this variability though, snow and ice dominate over the other hazards as the major closure mechanism.

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8. ANALYSIS OF MITIGATION OPTIONS

8.1 Introduction

In this chapter, mitigation options are assessed for their potential to limit the risk of the Desert Road being closed. The costs of each mitigation option are evaluated, as well as estimates of the effectiveness of each option to either lower the probability of the road closing, or limit closure durations. This information is used to find the benefit-cost ratio for each option.

The use of some mitigation options will be subject to various environmental and cultural considerations. This study looks at only the economic viability of mitigation options, to be used as part of a decision process. The environmental effects and social consequences of mitigation will be an integral part of any decision. Unfortunately the scope of this project means that these effects are not able to be assessed fully, only highlighting areas that may prove to be important. It is anticipated that the economic analysis of the options may be used as an indicator of options that require further consideration. In conjunction with appropriate analysis of the environmental and social effects, the economic evaluation will also provide information that will be useful when deciding which mitigation options are appropriate.

The range of mitigation options considered in this thesis is not a complete list of all possibilities. The intention of this analysis is to illustrate the way in which the assessment of mitigation options should be approached. Given this, the major mitigation options that are likely to be considered have been included where possible.

8.2 Analysis Approach

The risk of the Desert Road being closed is defined as the product of the probability of the road closing and the consequences of closure. The effectiveness of any mitigation option will then depend on its ability to either reduce the probability of the road closing, or limit the consequences of closure. In this case, the consequences are the cost to the New Zealand economy of road closures. This cost will primarily be a function of the closure duration and the availability of alternative routes.

The effect of mitigation options on both the probability and cost of road closures is complex. In many cases, the mitigation options will alter the response of the road network to the hazard event. The threshold event size required for road closure may increase, or the ability of road services to recover following the hazard event may improve. Because of these complexities, it is necessary to actually remodel the risk of road closure to find the effectiveness of the mitigation option. The new risk of

road closure is quantified by finding the new probability of closure and cost of closure following a hazard event with the mitigation option in place.

Each mitigation option will have costs associated with it. These costs will include both the capital cost of implementing the mitigation, and the ongoing cost of maintaining the mitigation.

The total risk of road closure with the mitigation in place is found by adding the capital and ongoing costs of the mitigation to the new cost of road closure. Comparing this new cost of closure with the existing costs incurred by the present levels of risk, will provide a measure of the viability of the mitigation option.

Some of the mitigation options that are considered have a range of effects on the performance of the road network. Effects may include changes to travelling speed, travel time, and accident rate along the route. Unless these changes directly relate to the risks of road closure, they are not considered in this study. For example, the analysis of a road realignment specifically designed to reduce accidents would include the benefits of accident reduction. If a bridge is seismically retrofitted, and in the course of that retrofit the alignment of the road is improved, it could be expected that traffic speeds may increase or the accident rate reduce. These benefits are not included in the analysis, as they are incidental to the aim of reducing the risk of the road being closed.

As well as having economic costs or benefits, mitigation options may also have social, political and environmental impacts. Weighing up of these impacts will be an important part of any decision as to which mitigation options are desirable. In this thesis, where possible, the more significant of these impacts have been highlighted, though no attempt has been made to quantify these impacts or to weight their importance in a decision. The aim of this thesis is to identify some of the economically viable options that are worthy of further study. This economic assessment will form part of the information used to make a final decision.

Many of the mitigation options assessed in this study have not been implemented before in New Zealand. Because of this, their effectiveness, cost, and the time required to implement them are very difficult to estimate. Relevant accounts from their use overseas have been utilised where possible. Experience from those familiar with the Desert Road has also been incorporated. It has been necessary though at times to make an “educated guess” about the effectiveness, cost, and timing of some of the options. These estimates are valid for the purposes of this study, where the priority is to examine the use of methodology, and highlight areas that may warrant further study. The author however would like to caution others from using these estimates for other purposes without reassessing their appropriateness.

8.3 Economic Evaluation

Economic evaluation of the mitigation options has been done in the spirit of the recommendations outlined in the Project Evaluation Manual (PEM) published by Transfund New Zealand in 1997.

Benefit-cost analysis involves identifying all of the costs and benefits of a project option over a certain analysis period, and selecting the most economically advantageous option. The financial analysis done for roading projects is similar to that done for any conventional business investment. What differs is that roading projects are done on behalf of the whole community, and are publicly funded. Assessing the costs and benefits for the whole of the community, from a national viewpoint, is thus appropriate. Conducting analyses from a national point of view, only total gains and losses to the New Zealand economy are considered. Transfers of benefits and losses, though noted as distributional consequences, are viewed as only a redistribution of benefit or loss, and are not explicitly incorporated.

Often, market prices for an asset will not reflect its true economic or national resource cost. Differences may be due to transfer payments within the economy such as taxes, duties, or subsidies. They can also be due to market imperfections caused by monopolies in the market, or the true cost of an activity not being shouldered (polluter pays). When conducting an analysis from a national viewpoint, it is necessary to price the use of resources to reflect the true cost of their use. This is called shadow pricing. The PEM specifies unit costs to be used for most things common in roading projects, such as the cost of travel time and vehicle operating costs. These unit costs specified in the PEM already account for the differences between market prices and national resource costs, and therefore do not require any further adjustment. Capital cost estimates for construction and maintenance used in the economic analysis are also exclusive of GST, as this is a tax that represents a transfer of funds from the purchaser to the government.

A higher value is placed on costs and benefits that occur in the near future than those that are many years away. If ten dollars, invested for one year, earns one dollar in interest over that year, then this implies that 10 dollars now has the same worth as eleven dollars in a years time. The PEM requires that analyses account for this reduced importance of costs and benefits over time, by applying discount factors to all costs and benefits that occur at a future time. The Treasury Department recommended in 1971 that all government agencies use a discount rate of 10% (Copeland, 1979). The discount factor to find the present worth of a one off single payment in some future time (single payment present worth factor, SPPWF) can be found as:

$$SPPWF_n^i = \frac{1}{(1+i)^n} = \frac{1}{1.10^n} \quad (8-1)$$

where i is the discount rate, and n is the number of years after time zero (time when the project is commenced) that the payment occurs. The present value (PV) of this single payment in some future time is then:

$$PV = SPPWF_n^i * \text{Cost or benefit at time } n \quad (8-2)$$

Where a series of equal costs or benefits arise each year of the analysis, or continuously over a period, the uniform series present worth factor (USPWF) can be used to calculate the present worth of these payments. This uniform series present worth factor assumes that the costs or benefits are accrued evenly throughout the year.

$$USPWF_n^i = \frac{(1 - (1+i)^{-n})}{\log_e(1+i)} \quad (8-3)$$

The present value of these annual payments is then found using Equation 8-4.

$$PV = \text{Annual cost or benefit} * (USPWF_e - USPWF_s) \quad (8-4)$$

where s is the year in which the annual payments start, or the first year that the benefit is realised, and e is the final year in which the payment is made or the benefit gained.

Some benefits and costs will increase arithmetically over time. Arithmetic growth assumes that costs or benefits are incurred continuously throughout the year and are compounding. Examples would be benefits and costs that are dependent upon the number of vehicles affected. Traffic is expected to grow arithmetically each year, so these benefits and costs would be expected to also grow arithmetically over time. The arithmetic growth present worth factor is found using Equation 8-5.

$$AGPWF_n^i = \left[\log_e(1+i) \right]^2 - n(1+i)^{-2} \left[\log_e(1+i) \right]^1 - (1+i)^{-n} \left[\log_e(1+i) \right]^2 \quad (8-5)$$

To find the present value of those costs and benefits that increase arithmetically, both the arithmetic growth present worth factor and the uniform series present worth factor are used.

$$PV = \text{Annual cost or benefit at time zero} * \left\{ (USPWF_e - USPWF_s) + R(AGPWF_e - AGPWF_s) \right\} \quad (8-6)$$

where R is the arithmetic growth rate at time zero.

In this study, the consequences of risk have been defined in terms of the expected duration of road closure and cost of road closures due to each hazard. The probability of each hazard occurring in any one year is also quantified. This means that the risk is then defined as the expected cost of closure in any one year. Costs and benefits of a mitigation option must then be projected over a 25 year analysis

period. The benefit of reducing road closures in future years will be dependent upon the number of trips that are disrupted by the road closure. This will in turn be dependent upon the expected growth in traffic. This relationship however is likely to be complicated by the fact that the cost of road closures may not increase linearly with the total number of trips made. Increased congestion on roads, particularly on the alternative routes when the Desert Road is closed, may inflate travel time required for detours and cause a disproportionate increase in the cost of road closure. As all the roads modelled in the network are presently uncongested, the increase in traffic flows would have to be substantial before this factor would come into play. It is therefore reasonable to assume that the costs of road closure would increase arithmetically with time, with an arithmetic growth rate of 0.05, the same as the traffic growth rate.

None of the natural hazards that have been modelled are time dependent. They are assumed to occur randomly in time, being no more likely to occur in any one year than another. This allows the probability of closure to be modelled as independent of the year of interest. Traffic accidents are an exception to this however, as they are dependent upon the density of traffic on a road, which is time dependent. To find the present worth of traffic accidents it was necessary to use the arithmetic growth rate for accidents found in Chapter 5.

All costs and benefits are expressed in terms of present day or constant dollars. For this study July 1997 dollars are used, as this was the financial year in which the evaluation was prepared. If this analysis were to be used in an application for funding, the application would be submitted for the 1999/2000 year of funding. This means that time zero for the projects would be 1 July 1999.

8.4 Mitigation Options

Mitigation options that were considered include:

- the purchase of specialised snow clearing blades to speed the removal of snow from the road surface;
- application of chemicals to the road surface to limit ice formation. Salt and Calcium Magnesium Acetate were considered, both with and without road weather information support;
- the use of Bailey bridges or the construction of fords past damaged bridges to re-establish traffic flow;
- seismic retrofit of the more vulnerable bridges along the Desert Road;
- realignment of a section of the Desert Road that is an accident black-spot to reduce the number and severity of accidents occurring;
- the use of variable message signs to better inform road users of detour options.

Details of the method used for analysis of each of these mitigation options are outlined in the following section.

8.4.1 Specialist Grader Equipment

In their 1996 report, Works Consultancy Services investigated the value of snow ploughs, or snow clearing blades, for use on the Desert Road. In that report, Works Consultancy Services recommended that a snow clearing blade be made a minimum resource requirement for snow clearing operations. Specialised snow clearing blades have been purchased, and were used on the Desert Road in the winter of 1997. These plant are expected to clear snow from the road surface faster than was previously possible. Hence the current risk of road closure due to snow and ice will be lower than that predicted using the 1994-1996 data. It is therefore necessary to estimate the new risk of closure that can be expected with this mitigation in place.

Snow clearing blades may be attached to the front of a 6-8 tonne truck, for the purpose of clearing snow at high speed. The attachment mechanism is a "quick-fix" type, allowing the blade to be attached to, or removed from, a parent vehicle as required within a matter of minutes. A blade that is set up correctly can be expected to clear snow depths of 600-700mm at speeds of between 40-50km/h. In the past, graders have been the common means of clearing snow on the Central Plateau. Blades mounted on these graders clear snow at a rate of only 10km/h.

Specialised snow clearing blades are used as a supplement to existing clearing procedures. Works Consultancy Services stated:

"It is not anticipated that the blades would replace the plant currently employed for these purposes but rather complement it by providing a system capable of clearing larger stretches of highway, while the conventional plant concentrated on the shorter winding sections."

The main advantage of specialised snow clearing blades would be faster clearing times. As it would be able to be used 4-5 times faster than conventional plant, it could be expected that clearing times would be at least halved. As time progresses, and there have been more winters in which the snow ploughs have been used, observed clearing times will provide a better estimate of the plant's effectiveness.

The operational costs of using the specialised snow clearing blades are estimated at \$75 per hour. This cost will be offset by the faster clearing times possible, and redundancy of other snow clearing plant that is no longer required.

Cost to purchase snow plough equipment

$$= 1 \times \$16,000 \text{ (Imported)} + 4 \times \$10,000 \text{ (Home Built)}$$

The specialised snow clearing blades may be used in conjunction with chemical methods of ice prevention or removal, speeding up the initial clearing, and ongoing removal of snow deposits from the pavement surface.

It is anticipated that the purchase of specialised snow clearing blades will also be advantageous in the event of a volcanic eruption. These blades may be used to clear deposits of ash off the road pavement both quicker and more effectively than plant that is presently available. Problems experienced after the Mount St. Helens eruption in Washington in 1980, with ash hampering parent vehicle use, are still

likely to be experienced though. In Washington, operators found that they needed to clear the air filters of parent vehicles every 100 miles or so (Foxworthy, 1982). The use of snow ploughs to clear ash was also often frustrated because ash drifts like snow but does not melt, often blowing back onto the road surface again. Even though snow ploughs proved to be limited in their effectiveness of clearing ash from roads, they could still be expected to be faster and more effective than current plant available for the task. Countering this though is the fact that road clearing plant has a limited impact on the duration of road closure following a volcanic eruption. Even with the road cleared, the atmosphere is still likely to be hostile to vehicle engines. This combined with the reduced visibility and accident risks is likely to dictate closure.

Benefit-Cost Analysis

Figure 8-1 shows the cumulative distribution of the benefit-cost ratios predicted by the snow and ice model for purchasing specialised grader equipment. The cumulative distribution shows that there is a 65% chance that purchasing specialised grader equipment will attain a benefit-cost ratio greater than the current incremental benefit-cost cut off ratio, which was set at 4.0 in August, 1998. This incremental benefit-cost cut off ratio is set to reflect the current budgetary restrictions of how many projects can be funded in any one year. The slope of the cumulative distribution reflects the high variability in the model predictions, with a benefit-cost ratio greater than 20 predicted to be attained 25% of the time.

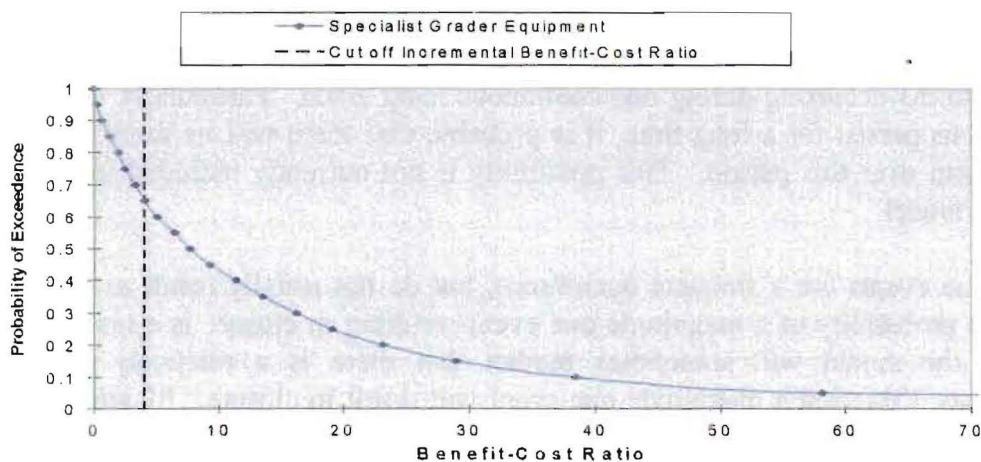


Figure 8-1: Exceedence probability distribution of the benefit-cost ratio predicted for purchasing specialist grader equipment

A major source of the high uncertainty for this and the other model predictions, is that although the benefit-cost ratios are calculated over a 25 year analysis period, the model generates a single average cost of closure for all years in the 25 year period. It should be noted that if the model were to generate an individual cost of closure for each year in the analysis period that this would reduce the uncertainty in the benefit-cost ratios predicted. Adding the results of each of the individual years would reduce the importance of years predicted to have extreme occurrences, and the overall benefit-cost ratio will tend towards the true benefit-cost ratio. Further work may be

beneficial to evaluate the extent to which this would reduce the overall uncertainty in the model predictions.

Using the computer package @RISK (Palisade Corporation) to simulate the benefit-cost ratios predicted, allowed the sensitivity of the model results to uncertainty of individual parameters to be analysed. This sensitivity analysis indicated parameters to which the model was especially dependent. Sensitivity analysis indicated that the benefit-cost ratio for purchasing specialist grader equipment is sensitive to the proportion of closure events that have snow as the major closure mechanism, as opposed to ice. The model was also sensitive to the duration of closures due to snow. What these results indicate is that these two factors that should be estimated with great care. With the available data it was only possible to make rough estimates for these parameters based on contractors impressions of the closure mechanisms. The large uncertainty in these parameter estimates translates through to the model prediction variability. In order to get better estimates, it would be necessary to track through the roading contractors' log books to find the actual causes of past closures, how often snow was a contributing factor, and how quickly that snow was cleared.

The sensitivity analysis also indicated that the benefit-cost ratio is especially dependent upon whether or not magnitude one events result in closure. This is a sensitivity that was not anticipated, and it highlights a deficiency in the closure model. As the model is presently set up, it assumes that any closure has a 40 percent chance of having snow as a causative mechanism. This is independent of the length of icing conditions that occur. In some senses this assumption is reasonable, as most of the very long closures are caused by the presence of ice (though they may be triggered by a snowfall). However it does not account for the probability of two or more snow storms occurring during one continuous icing event. Particularly when icing conditions persist for a long time, it is probable that there will be more than one snow storm over this period. This possibility is not currently included in the snow and ice model.

Magnitude one events are a frequent occurrence, but do not usually result in road closure. The probability of a magnitude one event resulting in closure is a random variable, so the model will sometimes predict that there is a relatively high probability (say 30%) that a magnitude one event will result in closure. Magnitude one events occur frequently (on average 7 times a year), so this scenario would predict that there will be two closures a year due to magnitude one events. There is a 40 percent chance that these closures will be due to snow. The model overestimates the importance of magnitude one events because it calculates that each event in the one simulation has the same probability of resulting in closure. This means that in simulations where magnitude one events have a high probability of resulting in closure, the model assumes that all 7 events have this chance of resulting in closure (Equation 8-7).

$$\text{No of events resulting in closure} = \text{No of events} * \text{Probability of closure} \quad (8-7)$$

In reality, each individual event will have its own probability of resulting in closure. This means that the actual number of events resulting in closure will be:

$$\text{No of events resulting in closure} = \sum (\text{Probability of each individual event resulting in closure}) \quad (8-8)$$

This same problem does not occur with larger magnitude events, as there are less of these events occurring in any one year (for example, there is only a 10 percent chance of a magnitude 12 event occurring in any one year). At present there is no simple solution to this problem, though this is an area that could be improved with future development of the model.

As specialised grader equipment has already been purchased for use on the Desert Road, assessment of other mitigation options to reduce snow and ice closures assumes that specialised grader equipment are in use.

8.4.2 Application of Chemicals

Salt

Sodium Chloride (salt) is the most common form of ice control used worldwide. Its predominance is primarily due to its availability and relatively low cost. Salt is spread on road surfaces following icing conditions to break the bond between the ice and the pavement surface by lowering the freezing point of water. Details of the chemical reaction facilitating this process are given in Works Consultancy Services, 1996. Once the bond between the ice and the pavement has been broken, any remaining ice can then be removed easily with snow clearing plant.

Salt is primarily used as a reactionary measure. It is applied during or after a storm event to break the ice to pavement bond once it has formed. The amount of salt required to break this bond has been estimated at 40g/m² (Perry and Symons, 1991). Assuming the width of the Desert Road is 9 metres, this translates to an application rate of 0.36 tonnes per kilometre.

There is some question as to the applicability of overseas data for the application rate of salt required. The effectiveness of chemicals applied to a road surface will be dependent upon the characteristics of the snow and ice present. Snow in New Zealand typically has a high moisture content. This may inhibit the effectiveness of any chemicals. Until there has been a trial of the use of these chemicals in New Zealand, the actual application rate required will be uncertain. This was taken into account by assuming the application rate is distributed normally about the average application rate (0.36 tonnes/km), with a standard deviation of 25% of this average (0.09 tonnes/km).

The likelihood of a given section of road being prone to icing will depend upon its surrounding environment. Factors such as shading of the road, its exposure to wind, and the local topography (i.e. a local depression) will all affect the susceptibility of the road to icing. Works Consultancy Services (1996) identified five key areas along the Desert Road that are vulnerable to icing, and generally cause the whole of the

Desert Road to be closed. The total length of the five sections is 9.23km, and it is this length that would benefit from salt application.

Kelly (1996) estimated the cost to supply salt to the Desert Road would be \$170/tonne, delivered to site. This means that the cost of supplying the salt would be:

$$\begin{aligned} &\text{Average cost to supply salt} \\ &= \$170/\text{tonne} * 0.36 (\pm 0.09) \text{ tonne/km} * 9.23 \text{ km} * N^{\circ} \text{ of applications per} \\ &\text{year.} \end{aligned} \quad (8-9)$$

The number of applications required in a year will depend upon the severity of the winter. The severity of the winter is in turn related to the number and duration of road closures that could be expected to occur in that year.

The frequency of application required (i.e. the number of hours before re-application is required) will be dependent upon the weather conditions, traffic density, and the need for the road surface to be ploughed. There is little information as to the frequency of application that would be required on the Desert Road. As an initial estimate, it shall be assumed that an application will be required once for every eight hours (± 3 hours) that the road would have been closed if the salt were not used.

Other costs associated with the application of salt on roads include the costs of spreading the salt, and the provision of washing bays to clean salt residue off cars. Kelly (1996) estimated those costs to be as follows:

Component Costs of spreading salt

Truck = \$60/hour
Spreading Gear = \$25/hour
Assistant labour = \$15/hour
Supervisor = \$25/hour
Loader = \$75/hour
Total Cost per hour = \$200/hour

Cost of Salt Washing Facilities

Construction costs = \$120,000
Operating costs = \$20,000/year
Maintenance Costs = \$10,000 in year 5

Application of salt in its dry form onto the road surface can be of limited effectiveness. The salt must dissolve before it can be of use. Studies have also shown that dry salt is more likely to spread to the verges of the road, bounce off the pavement surface when it is applied, and to be blown off the road by traffic. Pre-wetting the salt prior to application reduces these effects. Pre-wetting of the salt also allows application rates to be increased, with spreading speeds between 50 to 60km/h acceptable (Gustafson, 1993).

Assuming that spreading speeds of 50 to 60 km/h are applicable in New Zealand, the actual time for the spreader truck to apply the salt would be this speed multiplied by the length of section requiring treatment. The time taken for the plant to reach the

site and move between the treatment sections would also need to be included, as well as the time to load and prepare the chemical. If salt application is trialled, observed times will provide a better estimate of the time required. As an initial estimate it will be assumed that the plant is used for 2.5 (± 0.5) hours per application.

Costs of spreading salt

$$\text{Total Cost} = \$200/\text{hour} * 2.5 (\pm 0.5) \text{ hours application/day} * N^{\circ} \text{ of applications per year.} \quad (8-10)$$

If the salt is applied after ice has already formed on the pavement, it is likely that the road will need to be closed while the ice is cleared. Clearing time will include:

- the time to spread the salt;
- the time for the salt to break the ice to pavement bond;
- the time for the ice to be cleared from the road surface.

The duration of the road closure will be independent of the duration of the storm. The lag between recognising that ice has formed and its removal would only occur at the start of an icing event.

The time taken by salt to penetrate snowpack and break the bond between the ice and the road is usually within 30 minutes of application (Manning and Perchanok, 1993). After the bond is broken, it is possible to remove the ice using snow clearing plant. The time required to clear the road of ice once it has formed was estimated to be 4 (± 1) hours.

Although the application of chemicals should be able to prevent most closures due to ice on the road surface, closures could still occur due to heavy snowfalls on the road. These closures will mainly occur when the snow is falling at a rate too fast for snow ploughs to effectively keep the road clear. When this occurs, the snow depth will build up over time, potentially trapping vehicles if there is traffic still using the road. If vehicles do become trapped on the road, and night fall is approaching, then the salvage of the trapped vehicles may be delayed until the following day. This would be a worst case scenario, resulting in closures of up to 30 hours long. Typically, snow plough plant will be able to clear the road within three hours once the rate of snow fall has eased. This estimate assumes a clearing rate of 10km/hr, and that plant are working from both ends of the road. Adding on the time waiting for the snow to ease, the duration of closures due to snow alone have been tentatively estimated as 7 (± 4) hours. This time would be halved if specialised snow clearing plant were used. Closures due to snow alone, without the effects of ice, are estimated to account for approximately 40% ($\pm 10\%$) of all snow and ice closures.

The major criticism of using salt on roads is its environmental impact. When salt is applied to a highway, a proportion of salt migrates to the verges of the road or enters the natural drainage system. When salt is dissolved into solution it becomes dissociated into ions of sodium and chloride. Sodium ions are readily retained within the soil structure, decreasing soil permeability and lowering the soil fertility. High sodium and chlorine concentrations can also be toxic to some plant species. A large portion of the sodium chloride applied to a road will eventually end up in the

natural drainage system. Chlorine ions in the drainage system have a negative effect on aquatic life, being lethal in high concentrations. Sodium chloride is corrosive to vehicles, and if not washed off, can lead to car bodywork corrosion and shorter vehicle lifetimes. It is also corrosive to highway structures, with the chlorine ions migrating through capillary pores in concrete, corroding the reinforcing bars inside. This corrosion can be limited through maintenance and preventative construction measures such as increasing concrete cover over reinforcement.

More detail of the environmental effects that can be expected due to salt application on the Desert Road can be found in Works Consultancy Services (1996). It should be noted that the environmental effects of salt application are likely to be the key factor in determining its acceptability. An initial application for a resource consent to apply salt on the road without road weather information system support was withdrawn. Its acceptance seemed unlikely in the face of objections from the Automobile Association and the Department of Conservation regarding the negative effects of salt application.

Calcium Magnesium Acetate

Calcium Magnesium Acetate (CMA) is a formulation of dolomite lime and acetic acid presented in granular form. CMA does not melt ice or snow, but interferes with the molecule's ability to adhere to another molecule, or to the surface of the road (Works Consultancy Services, 1996). This keeps any snow fallen in a loose state, easing its removal by a snow plough or grader. It also breaks the bond between ice and the pavement.

CMA for use on the Desert Road would need to be imported to New Zealand, greatly increasing its supply cost. The cost to import CMA from Criotech in the United States is US\$1450/tonne, or NZ\$2,736/tonne (exchange rate at the time of the quotation US\$0.573=NZ\$1). The ratio by weight required to remove a given depth of ice is 1.4 to 1.7 times that required by salt (Manning and Perchanok, 1993). This would indicate an average spreading rate of 62g/m², or 0.56 tonne/km. Once again, there will be uncertainty as to the actual application rate required under New Zealand conditions. Studies in Ontario indicate that in short storms the quantity of CMA used was much higher than that of salt. An exception though was during longer storms, when CMA was relatively more effective, with residual effects observed from one storm to the next (Manning and Perchanok, 1993). Actual application rates and the timing of chemical application will need to be refined once trials of CMA are conducted under New Zealand conditions.

Average cost to supply CMA

*= \$2,740/tonne * 0.56 (± 0.14) tonne/km * 9.23 km * N^o of applications per year.*

(8-11)

The costs of spreading CMA are the same as those predicted for Sodium Chloride, but washing facilities would not be required.

Similar to salt, CMA applied as a reactionary measure, during or after a storm event, is likely to still require the road to be closed. The duration of this closure will depend upon several factors:

- the time required for the material to be spread;
- the time for it to break the ice to pavement bond;
- the time for the road to be cleared of the ice so that traffic flow can resume.

In a study conducted in Ontario (Manning and Perchanok, 1993) it was found that the times it takes to achieve bare pavement conditions with CMA and salt are very similar. These times will be independent of the storm duration, and is estimated to be 4 (± 1) hours per storm event.

The effectiveness of CMA in maintaining ice-free pavements significantly reduces at temperatures below -6°C . Above this temperature it performs to a level comparative with sodium chloride. As temperatures in New Zealand only rarely fall below this threshold, it is anticipated that CMA will be effective during most winter conditions on the Desert Road.

CMA does not have the negative environmental effects of sodium chloride. CMA biodegrades naturally, providing nutrients, helping soil permeability, and providing a source of carbon for micro-organisms. Most soils have the ability to immobilise the amounts of calcium and magnesium to be applied as CMA. The reaction required for this though can mobilise trace elements such as iron, zinc, and copper that are already present in the soil structure. It is unknown if this would pose an environmental threat along the Desert Road. Most of the CMA applied will be immobilised in the soil structure, but any residual that reached groundwater would increase water hardness. It can also deplete oxygen in closed systems when in high concentrations, though these conditions are not expected to be met at the Desert Road. CMA does not cause any corrosion problems.

The effectiveness of CMA in New Zealand conditions is expected to be trialled on the Desert Road in the upcoming winter of 1998. After CMA has been trialled, it will be possible to refine the estimates of the application rate required and costs.

Salt and Road Weather Information Systems

Road Weather Information Systems (RWIS) provide detailed information on real time pavement and atmospheric conditions. Data on the road surface temperature, wetness, the amount of residual chemical present on the road, as well as a summary of the existing weather conditions are all provided by RWIS. Combining all of this information allows a meteorologist to predict the likelihood of weather conditions deteriorating. Advanced warning of imminent bad weather conditions allows road managers to prepare for and be pro-active in the maintenance of road services. RWIS can also provide warning to motorists of hazardous driving conditions, with variable message signs indicating when ice is present on the road. Providing road managers with advanced warning of deteriorating weather allows chemicals such as sodium chloride to be spread in an anti-icing capacity. Less sodium chloride is required to prevent ice from bonding to a pavement than is required to break the

bond once it is already formed. Spreading rates of 10g/m^2 ($0.09\text{ tonne/km} \pm 25\%$) are required compared to the 40g/m^2 when sodium chloride is used as a reactive measure. In addition to limiting supply costs of the chemical, significant reduction in the environmental effects associated with the spreading of sodium chloride can also be expected.

Dravitzky and Varoy (1997) investigated the potential use of RWIS in New Zealand. They found that the cost of a single RWIS outstation with pavement sensors and meteorological sensors ranges from \$15,000 to \$40,000 (1994 NZD). In Finland, the average spacing of RWIS on rural roads is 25 km (Kulmala, 1998). This indicates that initially, one RWIS should be sufficient to provide information on pavement conditions at the 5 sites of interest.

Forecasts made from this one outstation, may then be interpolated over surrounding road sections using thermal mapping surveys (Thornes, 1989). Thermal mapping surveys are done using infra-red detectors driven over the road at night, finding the relative temperature profiles along the road section under different weather conditions. These allow the conditions at locations along the road to be predicted given the conditions at the outstation. It is not expected that the interpretation of the RWIS data will cost any more than present methods of weather monitoring.

Costs to implement RWIS

RWIS equipment costs = \$15,000-\$40,000

Cost of thermal mapping = \$50,000-\$70,000

Average cost to supply salt

*= \$170/tonne * $0.09 (\pm 0.02)\text{ tonne/km}$ * 9.23 km * N° of applications per year.*

(8-12)

Once again, it is assumed that salt is applied to the road surface every 8 hours (± 3 hours) that the road would have been closed if the chemical was not applied. In reality, applications may not need to be this frequent. The RWIS is able to detect residual salt on the pavement surface remaining from prior applications. Further application of salt would not be necessary until these salt levels fall below that required to inhibit icing.

The costs of applying salt to the road would be the same as detailed when salt is applied without the support of a RWIS.

It could be expected that ice on the pavement surface would never form if salt was applied effectively as an anti-icing agent. This would mean that all accidents in which ice was a contributory factor could be eliminated. Ice is at its most slippery around 0°C , as shown in Figure 8-2, meaning that the most hazardous driving conditions will occur just as ice is forming on the road surface. This is the time when drivers are least expecting it.

Mitigation that prevents ice from forming will be most effective in preventing these ice related accidents from occurring. Mitigation that treats icy conditions once they

have formed will be less effective, as the most dangerous time for drivers will have already past before the mitigation was activated.

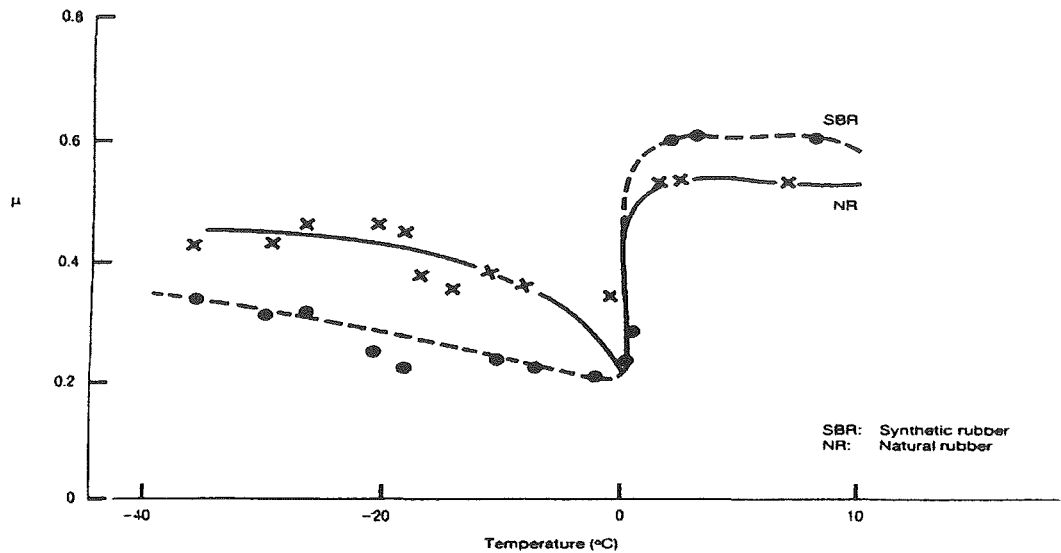


Figure 8-2: Skid resistance as a function of temperature (taken from Perry and Symons, 1991)

During heavy snowfalls, reduced traction due to the presence of snow lying on the road prior to ploughing could still be expected. This would mean that the hazardous conditions are not fully removed by the use of anti-icing agents. Snow is a far more visible hazard than ice however, and as such drivers are less likely to misjudge conditions. This model assumes that the number of accidents with snow, as opposed to ice, as a contributory factor is minimal. Virtually all accidents with ice or snow as a contributory factor will therefore be eliminated by the anti-icing mitigation. Chapter 6, Section 6.3.3 details how the proportion of accidents caused by ice were estimated.

With the implementation of RWIS, it is to be hoped that the stranding of vehicles, and long closure durations due to unexpected snow falls can be avoided. This is reflected in the model by a reduced variance in the duration of road closures due to snow.

Calcium Magnesium Acetate and Road Weather Information Systems

Literature indicates that application rates for chemicals used to prevent icing can be reduced to as little as 10 to 20 percent of the normal chemical application rate for deicing (Blackburn *et al.*, 1993). How much reduction is possible for CMA when used as an anti-icing agent though is unclear. It is assumed that the application rate is reduced by three-quarters of the original application rate without RWIS. The costs of the RWIS are the same as detailed earlier, but the new costs of supplying the CMA chemical would be:

Average cost to supply CMA

$$=\$2,740/\text{tonne} * 0.14 (\pm 0.04)\text{tonne/km} * 9.23 \text{ km} * N^{\circ} \text{ of applications per year} \quad (8-13)$$

Accident reductions that could be expected with the use of CMA as an anti-icing agent will be the same as those expected for salt and RWIS implementation. All accidents that have ice as a contributory factor would be prevented.

Benefit-Cost Analysis

Figure 8-3 shows the cumulative distributions of the benefit-cost ratios predicted for each of the chemical application mitigation options. What can be seen from this plot is that the application of salt onto the road surface to remove ice has a very high benefit-cost ratio (an average of 35). It should be emphasised that this benefit-cost ratio only indicates the economic effectiveness of salt to prevent the road from closing. It does not incorporate the environmental effects that would be caused by the salt, the magnitude of which are currently unacceptable in New Zealand (given that resource consents to apply salt have been denied). Use of salt with RWIS support attains an even higher benefit-cost ratio. This is due to the economic benefit of preventing accidents that have ice as a contributing factor. Using RWIS support will also reduce the amount of salt that needs to be applied to the road surface, significantly reducing the environmental impact of the chemical application. Both the economic and environmental benefits of using RWIS would justify its use if salt were to be applied to the road surface.

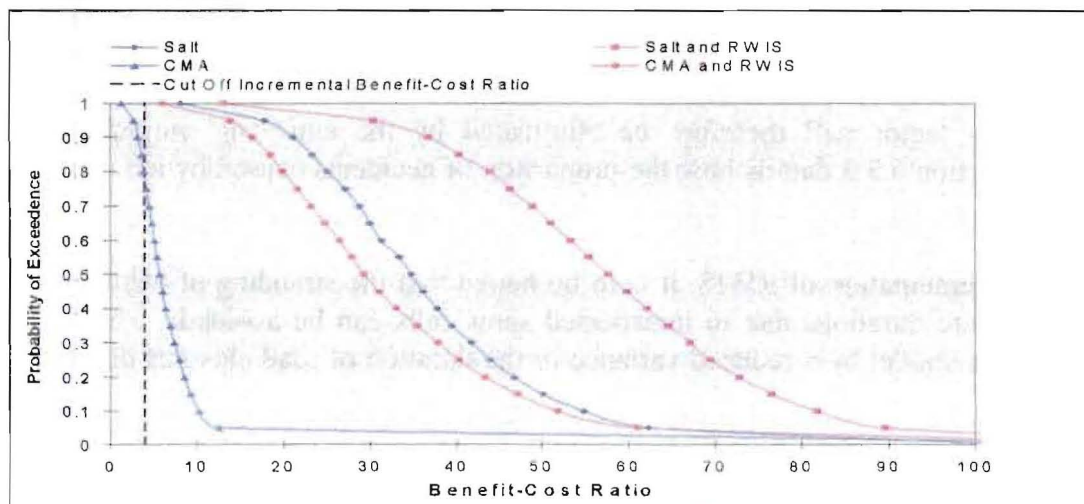


Figure 8-3: Comparison of the benefit-cost ratios attained for chemical application to the road surface

Even with RWIS support, the environmental effects of salt application may still be too high, making the use of CMA the favoured option. Again, the use of RWIS in conjunction with chemical application significantly enhances the benefit-cost ratio that is attained. This is due to two reasons. Firstly, with RWIS support, many ice related accidents can be prevented. This results in a significant saving to the New Zealand economy. Secondly, RWIS support enables the application rate of CMA to

be reduced. The cost to import CMA to New Zealand is very high, and any reduction in the amount that has to be imported will have a large impact on the benefit-cost ratio achieved. Capital cost savings to purchase CMA when using RWIS actually outweigh the cost to install and run the RWIS system. This means that using RWIS in conjunction with CMA is actually cheaper for Transit New Zealand than using CMA on its own.

Benefit-cost ratios predicted for applying CMA in conjunction with RWIS support range from 14 to 60 (5th and 95th percentiles). This means that even at the lower end of predictions, this is a very cost-effective mitigation option, definitely worthy of further investigation.

Sensitivity analysis of the model predictions indicated that the benefit-cost ratio attained is most sensitive to the application rate that is required for CMA. It is therefore this parameter that needs to be focused on when investigating the worth of applying CMA. The upcoming trial of CMA use on the Desert Road in the winter of 1998, should provide improved estimates of the actual application rate required for New Zealand conditions.

8.4.3 Temporary Access Past Damaged Bridges

Following an earthquake, any damage to bridges on the Desert Road will effectively close the road link until that damage can be repaired, or temporary access past the damaged bridges can be established. One option that was considered in Maffei's 1996 report for re-establishing traffic flow on a route with bridge damage, was to construct a temporary ford. It would take from 3-4 days to construct a ford that is capable of carrying heavy commercial vehicles.

Maffei provides some guidelines for estimating the costs of constructing and maintaining a ford, as a function of the in-place bridge length, and a terrain factor.

$$\begin{aligned} &\textit{Average cost to construct a ford} \\ &= [\$4,000 + \$200 * \textit{length of bridge (metres)}] * \textit{terrain factor}^2 \quad (8-14) \end{aligned}$$

$$\begin{aligned} &\textit{Average cost to maintain a ford per week} \\ &= [\$300 + \$4 * \textit{length of bridge (metres)}] \quad (8-15) \end{aligned}$$

The terrain factor accounts for the site conditions that may make it more difficult and expensive to construct a ford at certain locations. The terrain factor is taken to be 1.0 for most conditions, though bridges crossing very difficult terrain may warrant a factor of 1.2. Squaring of the terrain factor reflects the sensitivity of construction costs of a ford to the crossing conditions.

In the Project Evaluation Manual (Transfund New Zealand, 1997), guidance is provided on the costs for heavy commercial vehicles (HCV's) crossing fords. In this document it is estimated that the cost for a class 1 HCV to slow and negotiate a ford is \$1.13 (1994 dollars) per crossing. The cost to class 2 HCV's is \$2.03 (1994

dollars) per crossing. No estimate is given for the cost to private cars to cross a ford. Costs of negotiating a ford include:

- the time and vehicle costs of slowing to negotiate the ford;
- cost of travelling up and down the ford approaches;
- additional vehicle costs associated with the roughness of the ford;
- waiting time as vehicles queue to cross the ford.

To include the costs of cars crossing the ford, the approximate magnitude of these costs for all vehicle types was found. These costs were then factored by the typical vehicle composition on rural strategic roads, to give an average cost per vehicle negotiating the ford. This was estimated to be 46 cents per vehicle crossing.

Each ford will have differing dimensions, and the length of traffic queues will vary over time. To reflect these uncertainties, the model assumes that the true cost of negotiating each ford varies normally about 46 cents, with a standard deviation of 20%.

Cost of traffic slowing to negotiate ford

=46 ± 9 cents / vehicle crossing

Another option for restoring traffic flow may be to construct a Bailey bridge at the site. Particularly with the proximity of the Desert Road to the Waiouru Army Base, where there is expertise in constructing Bailey bridges, this may be a viable option. Bailey bridges were originally designed for military use, being quick and simple to erect. All parts of the bridge can be easily manhandled, and fit into 3-ton lorries. The bridges are strong enough to take vital transport and support weapons, and can be strengthened in-situ to carry heavier loads (War Office, 1956).

Average cost to construct a Bailey bridge, including bridge approaches

=\$300,000 (± \$30,000)

The bridge itself is likely to require very little maintenance. There will be costs involved though in maintaining the bridge approaches to a suitable standard. These are expected to be relatively small however, and will be independent of the bridge length.

Average cost to maintain a Bailey bridge

=\$100/week

Time taken to build a Bailey bridge

=1-2 days

Note that the time taken to actually construct the bridge is likely to only be 4-12 hours (War Office, 1956). The time assumed here includes time to get the required equipment to the site, and also time to develop approaches to the bridge that are suitable for sustained traffic flow.

In some areas along the Desert Road, the use of a Bailey bridge or a temporary ford will be restricted. This is primarily due to the local topography and the possible destruction of vegetation and land forms for the bridge or ford approaches. In these situations, the environmental effects of the temporary access are likely to prevent this mitigation option from being used, perhaps outweighing the costs of travellers using the alternative routes.

Less earthworks will be required for constructing a Bailey bridge than to construct a ford. Any earthworks will predominantly be in forming the bridge approaches, with little disruption caused to the actual river or stream bed. This compares to a ford construction, where there will be substantial earthworks involved, both in forming the ford approaches, and in developing the actual river crossing. The environmental effects of these earthworks to the aquatic environment in terms of increased sediment load are likely to be substantial. The location of many of the bridges within Tongariro National Park means that there is a heightened need to keep the environmental effects any mitigation to a minimum.

An advantage of building a Bailey bridge over constructing a ford is likely to be savings in the cost of traffic negotiating the temporary crossings. Less slowing would be required at a Bailey bridge (though some would still be required), the crossing is likely to be less rough, and there will not be the steep approaches required. These factors are likely to reduce crossing costs by approximately 50%.

Benefit-Cost Analysis

Figure 8-4 shows the cumulative frequency distributions of the benefit-cost ratios predicted for using Bailey bridges or fords in the aftermath of an earthquake. Although constructing fords to re-establish the transportation link has a very high benefit-cost ratio (an average of 275), the environmental effects of this mitigation are likely to prevent its use on the Desert Road. Bailey bridges, whilst not showing such a high benefit-cost ratio, are still extremely cost effective (an average benefit-cost ratio of 54). The reduced environmental effects of Bailey bridges means that this is likely to be the preferred option for the Desert Road.

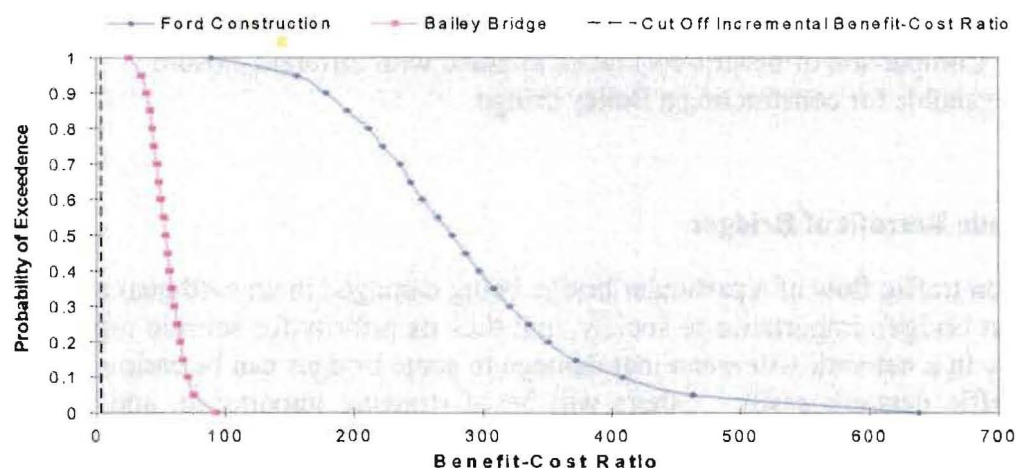


Figure 8-4: Probability of exceedence distribution of the benefit-cost ratios for constructing temporary access past damaged bridges

Sensitivity analysis of the benefit-cost ratio for using Bailey bridges indicates that the benefit-cost ratio is very dependent upon the percent damage to bridges during the smaller earthquake intensities. This sensitivity is due to the model assuming that a Bailey bridge will be constructed whenever there is bridge damage. Because of the relatively high construction costs of Bailey bridges (on average \$300,000 per bridge), it may be inappropriate to construct a Bailey bridge to provide temporary access if the duration of road closure is only going to be for a short time. This implies that there will be an optimum threshold closure duration, at which it becomes worth installing a Bailey bridge. The risk of closure model can be used to help determine what this closure threshold may be. Figure 8-5 shows the cumulative distributions of the benefit-cost ratios achieved when several different closure thresholds were trialled. It was found that the optimum threshold for installing Bailey bridges is for closure durations longer than 70 days.

As the construction of temporary access past damaged bridges has such a high cost effectiveness, it was assumed for the rest of this study that Bailey bridges will always be used where the closure duration is going to be longer than 70 days.

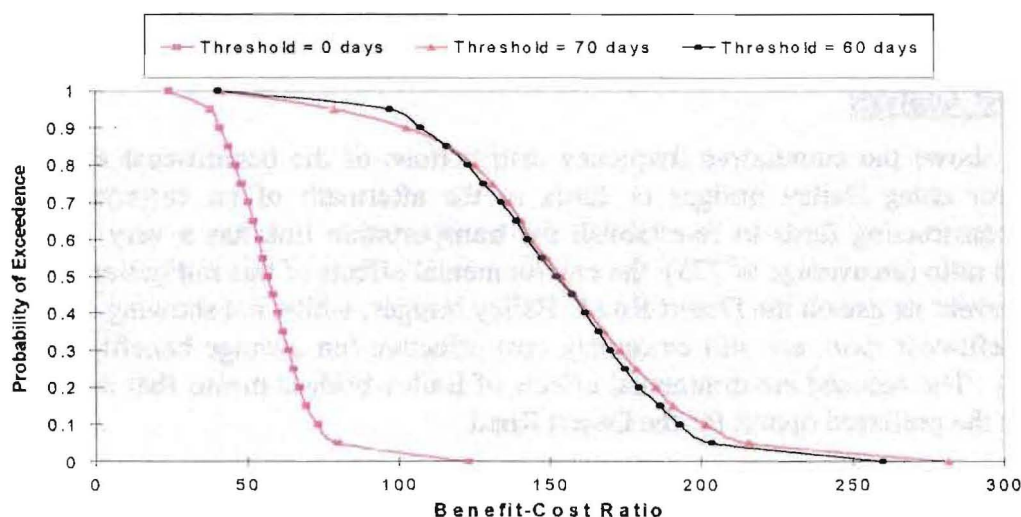


Figure 8-5: Comparison of benefit-cost ratios attained with different closure duration thresholds for constructing a Bailey bridge

8.4.4 Seismic Retrofit of Bridges

The effect on traffic flow of a particular bridge being damaged in an earthquake will indicate that bridge's importance to society, and thus its priority for seismic retrofit. Redundancy in a network will mean that damage to some bridges can be assimilated into the traffic network easily. Others will be of strategic importance, and their damage will severely impact on the ability of the network to provide the level of service required. The combined effect of the simultaneous loss of other bridges on a network will also determine the importance of a particular bridge in the system. Looking at the network as a whole, and the contribution of a particular bridge to that

network, is a much more valuable way of prioritising bridge retrofit (Werner *et al.*, 1997).

During an earthquake, damage to any of the bridges along the Desert Road, which makes that bridge unusable, will effectively cut the whole of the Desert Road link. The in-series nature of the bridges along the Desert Road dictates a strategy of retrofitting the most seismically vulnerable bridges first, aiming to bring all bridges along the road to a similar level of vulnerability. The level of this vulnerability will be determined by the risk aversion of those making decisions, and on the funds available.

It is of little use to spend large amounts of money upgrading one bridge along the route so that it will withstand very large earthquakes, if other bridges along the route are not up to this similar standard. The road will be closed by the weakest link in the chain. Having said this however, the time taken to resume services along a route will be affected by the number of bridges that have been damaged. It has been assumed in this study that the duration of road closure will be the time it takes to fix the bridge with the longest repair time, plus a quarter of the time that it takes to repair all of the other bridges along the route. This formula takes into account the fact that the repair of bridges in series will be hampered by access problems. Repairs to bridges at the middle of the route may not start until repair crews have established access past other bridges that have also been damaged.

The retrofitting of bridges may be carried out at two levels: the system level and the component level (Saiidi, 1992). The approach at the system level is to modify the structure so that the seismic forces experienced by the bridge are reduced. An example of this type of retrofit is the use of dynamic isolators at the superstructure-to-substructure connections. Retrofitting the components involves strengthening vulnerable parts of the bridge so that it can better withstand the shaking. Components that potentially need to be retrofitted include the piers, hinges, foundations, abutments, and superstructures.

In the most vulnerable bridge along the Desert Road, the Waihohonu Stream bridge, it is the column vulnerability that governs its vulnerability rating. The cause of this high vulnerability rating is that the piers for the bridge are reinforced concrete columns that were designed prior to 1973. Reinforced concrete columns designed prior to this time were not designed with ductile detailing. Past earthquakes have shown that these types of bridges are prone to catastrophic collapse (Maffei, 1996).

Column retrofit techniques include steel jacketing, active confinement by wire prestressing, use of composite material jackets (involving fibreglass, carbon fibre, or other fibres in an epoxy matrix), and jacketing with reinforced concrete. Of these, the most common retrofit technique that has been used to date has been steel jacketing (Priestley *et al.*, 1996).

Jackets act as passive confinement to the column, confining the concrete as it attempts to expand laterally. It reduces tensile stresses in the cover layer, thereby preventing spalling. Lateral expansion in the column is induced in the compression zone by high axial compression strains, or in the tension zone by dilation of lap

splices during splice failure. A similar lateral expansion in the column may also occur with the development of diagonal shear cracks (Priestley *et al.*, 1996). The confinement provided by the jacket changes the mode of failure of the column from a sudden catastrophic failure to a more desirable ductile failure.

Retrofit of the Thorndon Bridge in Wellington provides estimates of the cost to jacket vulnerable bridge columns in New Zealand conditions. Tender prices for the steel jacketing of the Thorndon bridge ranged from \$3500/tonne of steel plate to \$6000/tonne (1996 NZ\$). The price quoted tended to be dependent upon the economic conditions of the day, and the abundance of work available.

Expressing the price of retrofit by the tonne of steel plate used allows the cost of retrofitting a range of column sizes to be estimated. With typical jacket thicknesses of 10-12mm, and allowing for 25mm of grout between the column and jacket, the tonnage required can be calculated as follows:

Tonnes of Steel Jacket Required

$$\begin{aligned}
 &= \text{circumference of jacket} * \text{height of jacket} * \text{steel thickness} * \\
 &\quad \text{weight of steel} \\
 &= [\pi * (\text{diameter of the column} + 2 * \text{grout thickness})] * \text{height of} \\
 &\quad \text{jacket} * \text{thickness of steel} * \text{weight of steel} \\
 &= [\pi * (\text{diameter of the column} + 50\text{mm})] * \text{height of jacket} * \\
 &\quad (10 \rightarrow 12\text{mm}) * 7.85 \text{ tonnes/m}^3 \quad (8-16)
 \end{aligned}$$

The height of the steel jacket required will be dependent upon the type of seismic deficiencies within the column. If the column is suspected to be vulnerable to shear failure during seismic shaking, then the full height of the column will need to be jacketed. If there is a confining problem with the column, then only half of the column, either the top or the bottom, will need to be jacketed. Where the column has lap or curtailment deficiencies, the column needs to be jacketed for a height of twice the column diameter above and below the region of the deficiency only.

As the seismic vulnerability assessment was conducted to only a very preliminary level, the exact type of seismic deficiency of the columns has not been identified. At this stage of the model development, it is assumed that the whole height of the column is jacketed.

The unit weight of steel is 7.85 tonnes/m³. The cost to retrofit each column is found by multiplying the weight of steel used by the unit cost of the retrofit. This unit cost, expressed in terms of the tonnes of steel used, is an all encompassing cost of retrofit. It includes the costs of site measure, supply materials (steel plate and grout), fabrication (cut, roll, and corrosion protection coating), erection (including site welding and paint touch up) and grouting between the column and jacket.

During the Thorndon bridge retrofit, it was found that upgrading the bridge columns also necessitated the seismic retrofit of the pier caps. The seismically resistant columns were more likely to transfer seismic forces to the pier caps; seismic forces that the pier caps were not strong enough to withstand. The enforced additional retrofit of the pier caps significantly increased the cost of the bridge retrofit, with an

average cost of \$115,000 per pier. To account for this possibility, the cost to retrofit shall be weighted towards a more expensive unit cost. A triangular distribution of unit cost is assumed, with a minimum of \$3500/tonne, a mean of \$5000/tonne, and a maximum of \$6000/tonne of steel used.

Where movement joints are the critical feature of the bridges' vulnerability, there are two approaches that can be taken. Restrainers may be placed across the joint in an attempt to limit relative displacements, or the capacity of the joint to sustain displacement may be increased. With the first approach, the movement joints are locked so that no relative movement can occur. This is normally done by prestressing the frames together across the joint, or alternatively, oil viscous dampers can be used to connect the frames across the joint. Where locking of the movement joint is impractical or undesirable, such as at an abutment, the seating length can be extended to increase the displacement capacity of the joint. Extending the seating length is a comparatively inexpensive and simple procedure. Support lengths may be increased at abutments or under simply supported spans by attaching corbels or brackets. Where movement joints are located at positions other than at a support, direct seat extension is not possible. In this case it is necessary to provide an alternative support for the beam. This could take the form of an underslung beam to provide cantilever support, or thick walled pipe seat extenders that slide freely between the diaphragms (Priestley *et al.*, 1996).

The actual cost to retrofit a movement joint that is seismically deficient will be dependent upon the site conditions at the bridge. An abutment extension that requires earthworks, or the location of a pier in deep or fast flowing water, will increase the difficulty and thus the cost of retrofit. In general though, to retrofit a movement joint at an abutment is easier than retrofitting a movement joint at a pier. All of the bridges considered in this study that have movement joint deficiencies require retrofit at the abutments only.

The primary method of extending a seating length at an abutment in New Zealand is by fixing a concrete extension to the abutment. The cost of this concrete extension will be dependent upon the exact conditions at the site, but are estimated for a two lane bridge to be approximately \$10,000 per extension. To reflect the uncertainty of this estimate, the actual cost to be used in the model was expressed as a normal distribution with a mean of \$10,000 and a standard deviation of 20% of this mean. Cost estimates could be refined by estimating the quantities of formwork, reinforcing, concrete, etc. required for each individual bridge.

There is the potential for bridges to also be damaged in flood events. This damage could occur due to increased lateral pressures on the bridge when flood waters or debris pushes on the bridge piers and deck. Flood waters can also cause rapid scour around the bridge abutments or piers, which could also ultimately lead to the bridge failing. The streams and rivers that are crossed by the Desert Road are characterised by very low base flows, where flooding is unusual. Damage due to flood waters are therefore not likely to be significant. Lahars however, flowing down these river beds, could lead to bridge damage due to increased lateral forces or scour.

Lahars have a high sediment content, and thus would put large amounts of stress on bridge piers and on the deck of the bridge itself if the lahar reaches that level. Lahars also have a high scour potential, which may threaten bridge foundations. Very large lahars, such as would occur if there was a sector collapse at Mount Ruapehu, are likely to cover much of the Desert Road. If this were the case, damage to bridges due to the lahars would not be a primary cause of closure, and are thus not particularly significant. It is more likely to be middle sized lahars, flowing down existing river beds, under bridges and possibly damaging that bridge, that may require mitigation.

Beds of alluvial streams and rivers are subject to continual changes due to erosion and deposition of sediment. Any obstruction of the flow in these rivers or streams, such as bridge piers or abutments, will impact on this sediment transportation, and can lead to local scour developing at these obstructions. There are three types of scour that can affect a bridge structure; general scour, localised scour, and local scour (Dongol, 1989). Generalised scour occurs irrespective of the presence of a bridge, and is commonly known as the degradation or aggradation of the bed. Localised scour occurs where there is a constriction in the flow channel, forcing the velocity to increase. The increased kinetic energy of the flow increases its sediment load capacity and leads to degradation of the bed. Local scour occurs when there is an obstruction in the flow, such as a bridge pier or abutment. The local velocities caused by the obstruction cause scour to occur in the immediate vicinity of the obstruction.

Local scour can occur when the water is clear and there is no sediment being supplied to the scour hole. Under these conditions the limiting depth of a scour hole, determined by the geometry of the obstruction and the flow and bed properties, will be reached gradually over time. When flow velocities are higher, and there are large amounts of sediment transport occurring, live bed scour will occur. At this time, the scour hole will be supplied with sediment from upstream. For scouring to occur, the rate of deposition must be less than the rate of sediment removal from the scour hole. The limiting depth of the scour hole under these conditions is reached very rapidly.

Any scour to occur during a lahar would be characterised by live bed scour. As the sediment content of a lahar is very high, it could be expected that supplies of sediment to any potential scour hole would be enough to prevent significant scour from occurring. If scouring were to occur though, the scour holes would form very rapidly. Failure of the bridge may be both sudden and catastrophic. Preventative treatment would therefore need to be in place prior to any lahar forming to be of any use. Debris carried down by the lahar that may catch on the bridge piers will increase the depth of the scour hole that could be expected.

One possibility for preventing scour from occurring at bridge piers would be to use riprap on the river bed near the piers and abutments (Croad, 1997). Riprap essentially provides a protective barrier of immovable bed around the pier. As this protective bed cannot be entrained by the flow and removed from the locality of the obstruction, in theory there will be no local scour.

A more likely scenario of bridge failure during a lahar would be that of the bridge failing to withstand the large lateral forces imposed on it by the lahar. The large flows, and high specific gravity of the lahar fluid would impose large lateral forces on any obstruction to the flow, particularly the bridge piers or superstructure. It would be possible to strengthen bridges to help them withstand such forces, though the worth of this would depend upon the relative risk of a lahar occurring. As initial estimates indicate that medium sized lahars are not a significant closure mechanism, possible retrofit techniques to increase the lateral strength of bridges have not been investigated.

Benefit-Cost Analysis

Benefit-cost analysis was done for retrofitting the most vulnerable bridge on the Desert Road, the Waihohonu bridge, which currently has a vulnerability rating of 10. The Waihohonu bridge scores this rating because it has concrete columns at its piers, and these concrete columns were designed before 1973. This means that the columns may have non-ductile detailing, and be susceptible to catastrophic collapse in an earthquake. To retrofit the Waihohonu bridge, it is proposed that steel jackets be placed around each of the four pier columns to provide lateral support. Once retrofitted, the vulnerability rating for the Waihohonu bridge would reduce to 6.9. The cost of this retrofit is estimated at just under \$22,000.

Retrofit of the Waihohonu bridge would reduce the duration that the Desert Road is expected to close due to earthquakes, particularly following intensity 7 earthquakes. As well as reducing the total duration of road closure that is expected, retrofit of the Waihohonu bridge will also reduce the number of Bailey bridges that will need to be constructed. This will result in a capital cost saving to Transit New Zealand for the construction and maintenance of Bailey bridges.

In order to find the benefit-cost ratio for retrofitting the Waihohonu bridge, firstly the duration and cost of closures with the retrofit in place was assessed. This required re-assessing the closure threshold for constructing a Bailey bridge, with the new bridge vulnerability and closure scenarios. Once again, the optimum threshold for constructing a Bailey bridge was a closure duration greater than 70 days. The reader should note that the value of this threshold for constructing Bailey bridges will be specific for the combination of bridges along the Desert Road.

If the Waihohonu bridge were to be retrofitted, this would result in less damage to the Waihohonu bridge during any given earthquake, and fewer Bailey bridges being required. On average, it is predicted that this reduced expenditure on Bailey bridge deployment and maintenance will outweigh the cost of retrofit by \$57,000. In fact, the model predicts (with 95% confidence) that the savings in Bailey bridge costs will always outweigh the cost of the retrofit. This results in a net negative capital cost to Transit New Zealand to do the retrofit. To place this result within a benefit-cost ratio framework, the retrofit is awarded a benefit-cost ratio of 99. This will ensure its priority in funding allocations.

It is important to remember that this benefit-cost ratio was determined using only very preliminary estimates of the vulnerability of the Waihohonu bridge. Rather than interpreting this high benefit-cost ratio as an indication that the retrofit should immediately proceed, it should instead be viewed as an indication that the seismic vulnerability and worth of retrofitting the Waihohonu bridge is deserving of further investigation.

8.4.5 Road Realignment for Accident Reduction

As road accidents have the potential to close a road, any project that potentially reduces the number of accidents to occur will also reduce the risk of road closure. In this section, the magnitude of this potential benefit shall be assessed using (as a case study) a project that has been proposed for the Desert Road. This project has already been initially investigated to find its costs and benefits in terms of accident reduction, and impact on road user costs. Any reduction in accidents will have, as a by-product, a concurrent reduction in the risk of road closure. This is a tangible benefit that can be incorporated into the benefit-cost ratio for the project.

The accident reduction proposal that shall be used as a case study, is realignment of a 4.2 km section of the Desert Road, between Waikato Stream and Wharepu Stream (route positions 691/11.43 to 691/15.61). At present, this section of road has a poor vertical and horizontal alignment. This is out of character with the high standard of the rest of the Desert Road. Because the quality of this section of road is out of character, a large number of accidents involving loss-of-control tend to occur, primarily due to drivers misjudging these unexpected conditions. The proposed project would improve the vertical and horizontal alignment of the road, bringing it to a standard more in line with that elsewhere on the Desert Road. The proposed new alignment of the road is shown in Appendix G. In 1991, in the interim before the realignment could be done, "Reduce Speed" signs were erected to warn drivers that the area is a significant accident black spot. These signs have been very successful in lowering the number of accidents in the area, though some feel that their effectiveness may reduce with time.

The PEM recommends that evaluation of accidents be done using the accident history for the site from the previous 5 years. This would mean that any analysis done now would only reflect the reduced accident rate since the Reduce Speed signs were erected. If these were to reduce in effectiveness over time, then the accident history used would underestimate the number of accidents that could be expected in the future. For this reason, the analysis of the costs and benefits of the alignment has been done using an accident history of 10 years (Bloxam, Burnett and Olliver, 1998). This 10 year history spans the 5 years prior to and after the erection of the warning signs. The costs and benefits that have been estimated for the proposed project (excluding road closure costs) are tabulated in Table 8-1.

Table 8-1: Estimated costs and benefits for the proposed road realignment

	Do Minimum	Proposed Realignment	Net Costs/Benefits
Benefit Calculations:			
-vehicle operation costs	\$18,490,766	\$15,893,975	\$2,596,790
-travel time costs	\$12,444,384	\$9,792,756	\$2,651,627
-accident costs	\$28,156,145	\$8,680,578	\$19,475,567
-CO ₂ emissions	\$832,084	\$715,229	\$116,856
TOTAL BENEFITS			\$24,840,841
Cost Calculations:			
-capital costs	\$0	\$5,911,488	\$5,911,488
-maintenance costs	\$741,547	\$663,188	\$-78,359
TOTAL COSTS			\$5,833,130
BENEFIT/COST RATIO			4.3

The accident rate is only expected to change between the 'do minimum' scenario and the realignment scenario, for the section of road where the realignment takes place. It is therefore only necessary to assess the reduction of risk for this section, to find the total risk of closure when the realignment is complete. The total risk of closure due to traffic accidents will be the existing risk of road closure minus the reduction attained by the mitigation.

With the realignment completed, it is expected that the accident rate on that section of road will reduce to that typical for flat terrain with an AADT between 2500 and 12000 vehicles per day. This is a reported accident rate of 18 injury accidents per 10⁸ vehicle kilometres travelled. Over the 4.2 kilometre section of interest, with an AADT of 3,600 vehicles per day, this translates to 0.993 injury accidents occurring per year on that section of road. The duration of closure that results from these accidents will be dependent upon the accident severity and the type of vehicles involved. In Chapter 5, the proportion of accidents falling into each accident severity and vehicle type category were found for the Desert Road (Table 5-3). If this section of road is brought up to the standard found along the rest of the Desert Road, then it could also be expected that any accidents would be of a similar nature to that on the rest of the road. The number and type of accidents expected with the realignment in place is shown in Table 8-2.

When calculating the risk of road closure due to traffic accidents in Chapter 5, the accident record for the five years 1991 to 1996 were used. To retain consistency, this time period shall also be used to find the number and type of accidents that are expected to occur on the section of road under the 'do minimum' scenario. The number of accidents expected each year of each accident type is given in Table 8-2.

As the analysis by Bloxam, Burnett and Olliver was done using a 10 year accident history, for the sake of comparison, the number of accidents predicted per year using this analysis period is also shown in Table 8-2.

Table 8-2: The number of accidents expected to occur annually on the section of the Desert Road from Waikato Stream to Wharepu

Accident Type	'Do Minimum' N° of accidents/year		Predicted N° of accidents per year after realignment
	5 year analysis period	10 year analysis period	
Single Car, Non-Injury	1.2	1.4	0.47
Single Car, Injury	2.4	2.2	0.43
Single Car, Fatality	0.2	0.5	0.03
Multiple Car, Non-Injury	0.0	0.0	0.17
Multiple Car, Injury	0.2	0.2	0.27
Multiple Car, Fatality	0.2	0.2	0.05
Single Truck, Non-Injury	0.6	0.4	0.10
Single Truck, Injury	0.6	0.6	0.09
Single Truck, Fatality	0.0	0.0	0.00
Multiple Truck, Non-Injury	0.2	0.1	0.11
Multiple Truck, Injury	0.0	0.0	0.10
Multiple Truck, Fatality	0.0	0.2	0.03

Benefit-Cost Analysis

Realignment of the Waikato Stream section of the Desert Road results in an average reduction in road closure costs of \$18,600 per annum. As can be seen from Figure 8-6, this reduction in road closure costs is relatively insignificant to the benefit-cost ratio attained by the project. The majority of this project’s benefits arise from savings in accident costs, which is the primary aim of this mitigation. The project has a 69% chance of exceeding the cut off incremental benefit-cost ratio using traditional analysis, compared to a 73% chance when road closure effects are included.

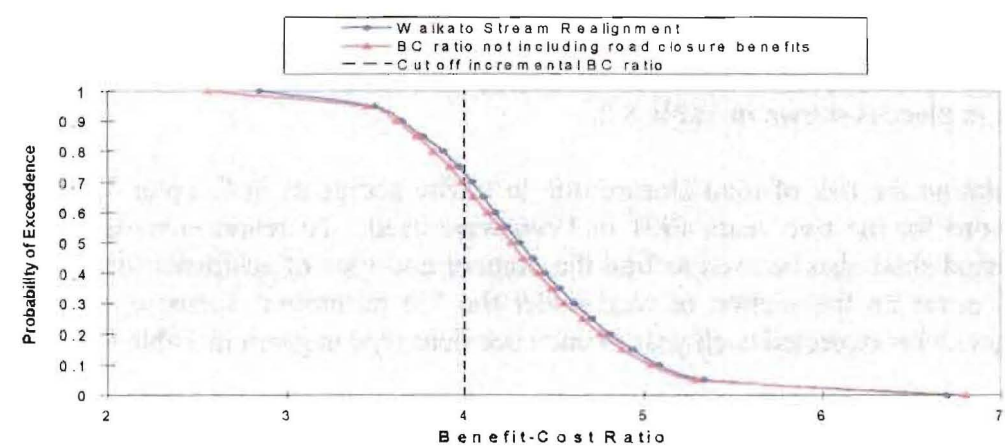


Figure 8-6: Comparison of the benefit-cost ratios calculated for the Waikato Stream realignment when reduced road closures are included in the analysis or excluded

Sensitivity analysis of the Waikato Stream realignment benefit-cost ratio indicates that the most important variable for this project is the capital cost estimate for actually doing the realignment. Variability in the capital cost estimate accounts for a large proportion of the spread in the cumulative distribution for the benefit-cost ratio (Figure 8-6). The benefit cost ratio is also sensitive to the savings in accident cost that can be expected with the realignment, and to the saving in vehicle travel time. The benefit-cost ratio is relatively insensitive to variability in the assumed road closure costs.

8.4.6 Variable Message Signs

Information regarding the availability of alternative routes provided to drivers will improve their ability to make decisions as to their best possible route to take through the network. This is particularly important when there are significant delays in the network, or if a road link is closed to traffic. For this information to be useful for drivers, and for drivers to take heed of the information, it must be both timely and perceived to be accurate.

If information on road closures is provided to drivers early enough in their journey, then they can divert to other routes in the network not affected by delays or closures. For example, if both the Desert Road and State Highway 4 are closed, north-south drivers can divert through either New Plymouth or Napier. For this to happen though, drivers must be aware of the closures early enough in their journey. Information on possible detours early in the trip limits the probability of drivers having to back-track to get onto these detours.

At present, the communication of road information is done via a network of manually operated sign boards distributed about the extended Central North Island. The information provided on these signs is a route description and a changeable open/closed sign blade. During any road closure, it is the responsibility of maintenance personnel for the area in which the sign is located to update the road status on the sign. Problems with this current system include its labour intensity, and time lags between a road opening and closing and the sign being changed to advise of the new road status. These delays can cause frustration for drivers, and a lack of trust of the information provided.

It has been proposed that an upgrade of the current system consider the use of variable message electronic signage (VMS) (Gordon, 1995). VMS can display any numerical or alphabetical message, and can be manually changed via a central computer linked to a telephone system. The signs on a network can be linked so that they are all changed by a single operator, or can be operated separately to provide information specific to that location. VMS can also be programmed using environmental sensors, to automatically display warnings if there is a change in environmental conditions, such as temperatures below that required for icing. Preliminary estimates of the costs to upgrade nine strategic signs throughout the North Island to VMS standard add to \$320,000.

When drivers have better information about which routes are available, they also have a greater ability to make decisions regarding route alternatives. When this information is provided early in the journey, drivers can modify their route to avoid the affected region. Drivers armed with this information are more likely to take alternative routes, rather than cancelling journeys, or waiting until their route of first choice becomes available again. Providing drivers with information on the availability of alternative routes will thus decrease the elasticity of driver behaviour.

The elasticity of driver behaviour describes the willingness of a driver to alter their travel plans, based on what the trip will cost (Equation 8-17)

$$\text{Elasticity} = \frac{(\% \text{ change in number of trips made})}{(\% \text{ change in the price of a trip})} \quad (8-17)$$

As an alternative route is not the driver's first choice, it is assumed that travel on the alternative route is more expensive than the route of first choice. If drivers have better information about the availability of the alternative routes, they are more likely to accept the higher cost of travel, and use the alternative route. Drivers will behave in a less elastic manner. Conversely, where drivers do not know about alternative routes, they are more likely to delay or cancel planned trips. In this case, as the cost of the trip increases (the cost to use the alternative route), many drivers will cancel or delay trips. Thus there is a proportionally large change in the number of trips made, given the change in cost. Drivers in this scenario are expressing very elastic behaviour.

Unfortunately there is little information available as to the elasticity of driver behaviour in New Zealand. Continuous traffic counts taken during the 9 day closure of 1995, were used in Chapter 7 to find the elasticity expressed by drivers in the Central North Island. These traffic counts indicated that on average drivers using the Central North Island network have an elasticity of travel of -3.0. This means the percentage of trips cancelled increases three times faster than the increase in the price of making the trip (Equation 8-18).

$$\begin{aligned} &\text{Elasticity expressed during 1995 closure} \\ &= \frac{(\% \text{ change in travel cost when Desert Road is closed})}{(\% \text{ change in number of trips made})} \\ &= -3.0 \end{aligned} \quad (8-18)$$

Changes to the elasticity of driver behaviour will alter the costs of road closures to the New Zealand economy. Better driver decisions will result in lower costs with fewer delays and cancelled trips. The increased propensity of drivers to divert to alternative, longer routes though will increase road user costs, and will extend road users' exposure to potential traffic accidents.

The amount that drivers' elasticity will change with the implementation of variable message signs is highly uncertain. Hanbali and Kuemmel (1993) found that there is typically a significant reduction in the number of trips made during hazardous

conditions such as winter storms. Factors affecting the number of trips made can be categorised into the following:

- a generating factor relating to the individual trip makers and their willingness to travel;
- an attraction factor related to the importance or utility of the particular destination;
- a linkage factor related to the difficulty or cost of moving from the origin to the destination;
- and other related factors.

These factors indicate that the elasticity of a particular trip will be dependent upon the reason for the trip, the perceived cost, and the element of danger involved with making the trip. Hazards that cause a road to close will also alter the willingness of a driver to travel. For example, following a major earthquake it is likely that many non-urgent trips will be cancelled as people stay at home to repair damage. Thus, even if alternative routes are available, people may still not be willing to travel. Similarly, road closures due to snow and ice may deter people from travelling to recreational sites, as the weather conditions causing the road to close also cause cancellation of the recreation. During winter conditions the perceived risk of accidents on the unfamiliar icy roads may also deter.

It can be seen from Figure 8-7 that as the elasticity of travellers within the Central North Island network gets closer to zero, the cost of road closures reduces. This implies that people are cancelling their trips when this may not truly be the most economic option. For this to occur, there must be other factors that are influencing driver behaviour other than the economic cost and benefit of the trip. These factors are likely to include the general environmental conditions that have induced the road closure (e.g. snow and ice or an earthquake), and the drivers imperfect knowledge of the road network and of diversion alternatives. Table 8-3 compares the cost of different road closure scenarios if driver elasticity can be brought closer to zero.

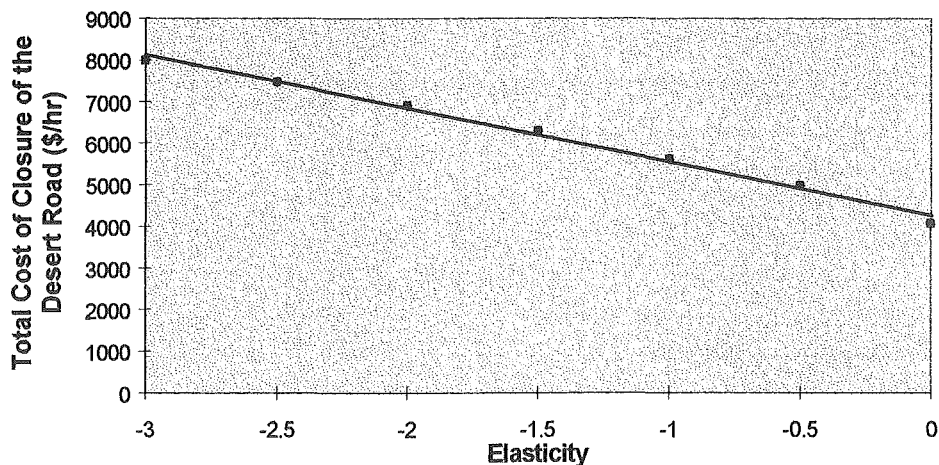


Figure 8-7: Sensitivity of the cost of closure of the Desert Road to the assumed elasticity of travellers

Table 8-3: Comparison of the total cost of closure scenarios with different driver elasticities

Closure Scenario	Total Cost of Closure (\$/hr)		
	Elasticity Parameter = 3.0	Elasticity Parameter = 2.0	Elasticity Parameter = 1.0
Desert Road Closed	7,990	6,900	5,620
D Rd ¹ and SH 1 ² Closed	12,570	11,230	9,630
D Rd and SH 4 ³ Closed	22,870	21,410	19,620
D Rd and SH 41 ⁵ Closed	9,240	7,970	6,490
D Rd and SH 47 Closed	14,350	12,990	11,260
D Rd and SH 49 Closed	12,180	10,860	9,310
D Rd, SH 1, and SH 49 Closed	11,230	9,930	8,383
D Rd, SH 4, and SH 41 ⁵ Closed	24,110	22,480	20,520
D Rd, SH 4, and SH 47 Closed	22,690	21,160	19,290
D Rd, SH 4 and SH 49 Closed	23,620	25,150	20,355
D Rd, SH 41 ⁵ , and SH 47 Closed	20,526	21,024	17,140
D Rd, SH 47, and SH 49 Closed	16,460	14,930	13,030
D Rd, SH 1, SH 49, and SH 4 Closed	22,630	21,160	19,374
D Rd, SH 4, SH 41 ⁵ , and SH 47 Closed	27,970	26,220	24,080
D Rd, SH 4, SH 41 ⁵ , and SH 49 Closed	24,850	23,220	21,260
D Rd, SH 4, SH 47 and SH 49 Closed	23,430	21,890	20,020
D Rd, SH 41 ⁵ , SH 47, and SH 49 Closed	23,640	21,950	19,870
D Rd, SH 1, SH 49, SH 4, and SH 47 Closed	21,085	19,550	17,685
D Rd, SH 4, SH 47, SH 49 and SH 41 ⁴ Closed	31,060	29,220	26,970
D Rd, SH 4, SH 47, SH 49 and SH 41 ⁵ Closed	29,950	28,200	26,041
D Rd, SH 41 ⁵ , and SH 49 Closed	13,430	11,940	10,190
D Rd, SH 1, SH 49, SH 4, SH 47, and SH 41 ⁵ Closed	27,970	26,110	23,822

¹ D Rd stands for the Desert Road.

² Taihape to Waiouru section of SH 1.

³ Raetihi to National Park section of SH 4.

⁴ Manunui to Turangi section of SH 41.

⁵ Turangi to the SH 32 section of SH 41

Installation of a VMS system in the Central North Island network could help to better inform drivers of diversion alternatives, and limit some unnecessary trip cancellations. Even with a perfect understanding of the road network however, some drivers will still be inclined to cancel their intended trip. The amount that the elasticity could be ultimately reduced to with a VMS system is very uncertain. There are few studies looking at the elasticity of drivers in rural uncongested networks, meaning that there are also few indications of what a reasonable level of elasticity would be.

It is unlikely that using VMS on its own will be very effective in reducing the elasticity of drivers travelling on the Central North Island traffic network. Installation of VMS would need to be supplemented with extensive public education campaigns, to ensure that drivers are able to use the information provided by the VMS to its maximum benefit. Drivers would need to be made aware that the alternative routes exist, and the additional time and distances required to use them. Only then will driver be able to make a fully informed choice as to whether to cancel their trip or not. Again, just how effective such an education campaign would be is very uncertain.

Benefit-Cost Analysis

Because of the uncertainty surrounding the effectiveness of VMS and public education campaigns to reduce driver elasticity, it is difficult to predict the benefit-cost ratio of such measures. The benefit-cost ratio will depend upon how much reduction in elasticity is attained from a given expenditure. This will not be known without further study into how much elasticity can be reduced, and the type of measures required to bring this about.

To provide an indication of the type of benefit-cost ratios that may be obtained, several trials were run to find the maximum expenditure on VMS and ongoing publicity campaigns, and yet still be accepted for funding. Figure 8-8 shows the benefit-cost distributions attained for various spending levels on public education, assuming that an elasticity of 2.0 is achieved. These trials assume that VMS is installed at an initial cost of \$320,000. To achieve a benefit-cost ratio of 4 or greater, 95% of the time, the maximum amount that may be spent on public education to reduce the elasticity to 2.0, is \$30,000 per year. If research shows that this elasticity reduction is attainable at an annual cost of \$30,000 or less, then this mitigation option becomes worthwhile.

Figure 8-9 shows similar trials of the maximum expenditure permitted to reduce elasticity to 1.0. These results indicate that if it is possible to reduce driver elasticity to 1.0, while spending less than \$100,000 per year, then this project may be worth funding. It should be noted that an elasticity reduction from 3.0 to 1.0 is a large change. In practice it may not be possible to reduce driver elasticity to this level.

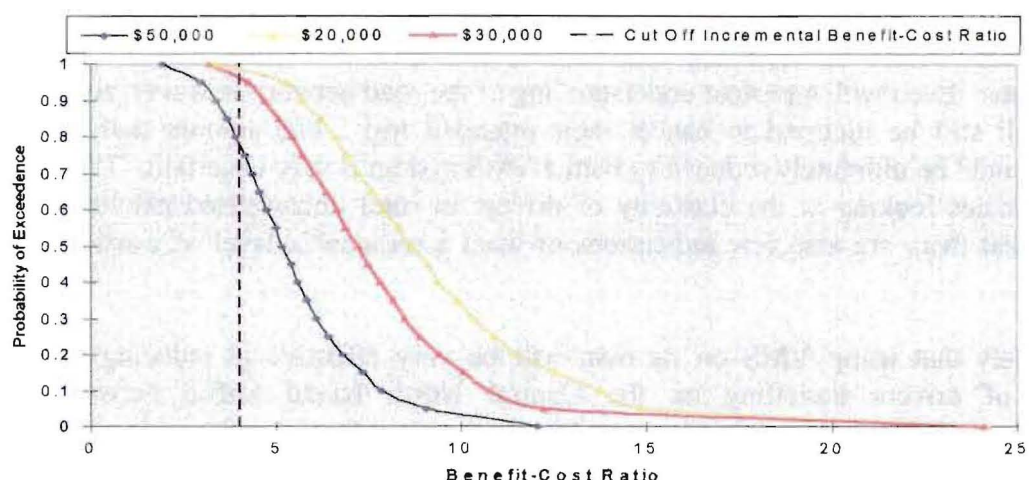


Figure 8-8: Comparison of benefit-cost ratios attained for different spending levels, assuming that elasticity is reduced to 2.0

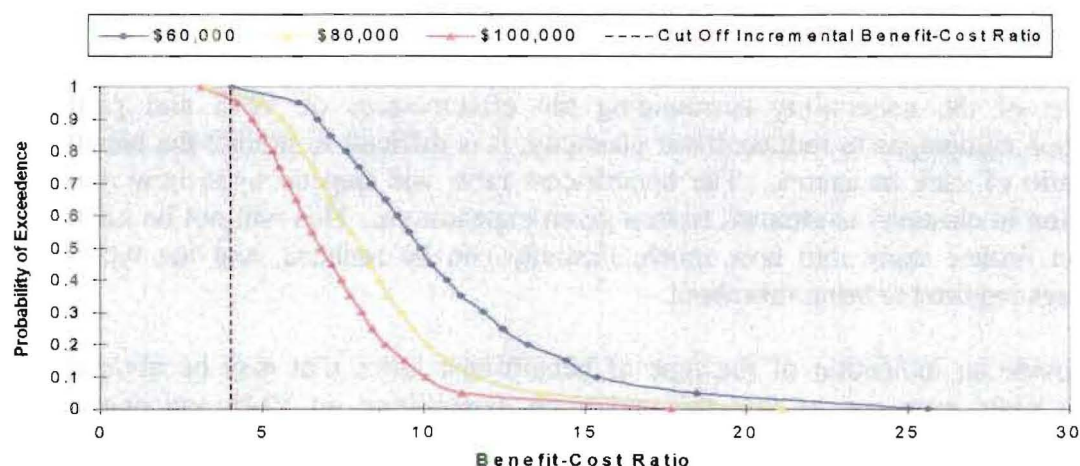


Figure 8-9: Comparison of benefit-cost ratios attained for different spending levels, assuming that elasticity is reduced to 1.0

8.5 Conclusions

The mitigation options investigated in this chapter do not constitute an exhaustive list of all possibilities. Neither is the detail to which they were evaluated sufficient to constitute conclusive evidence of an option's worth. The aim of this study was to illustrate an approach that could be used in the evaluation of risk reduction options. Many of the options considered have not been used before in New Zealand conditions, and estimates of their effectiveness are therefore inherently uncertain. The study does however highlight mitigation options that are worthy of further consideration, and for those options, indicate areas that would benefit from more in-depth investigation.

Of the mitigation options considered to reduce the risk of road closure, those that particularly warrant further investigation include the use of CMA and RWIS to reduce snow and ice closures; erection of Bailey bridges to re-establish flow past bridges damaged in an earthquake; and the retrofit of seismically vulnerable bridges along the Desert Road.

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9. OPTIMUM DISTRIBUTION OF RESOURCES

9.1 Introduction

Deciding which mitigation options are worthwhile, and how many resources should be committed to each can be difficult. The decision becomes even more perplexing in situations such as that presented by the Desert Road, where the failure mechanisms are complex, and the possible mitigation options are numerous and interacting. The large amounts of information available seems to be almost a disadvantage, acting to confuse rather than enlighten those faced with making decisions. It is with this background that a computer program has been developed, to sort the available information into a form that is both understandable and manageable.

The program takes into account the effectiveness of different mitigation methods in reducing the probabilities and costs of the closure mechanisms, and the interdependencies between these mitigation options. The effect of uncertainty in the available information on the optimum resource distribution is also assessed.

It is envisaged that this prototype will have wider applications, providing a tool that is useful in making various complex decisions where the effectiveness of a limited resource needs to be optimised.

9.2 Program Development

9.2.1 Optimisation Method

Initial theory for optimising resource distribution for maximum risk reduction was developed by Elms (1979, 1981, 1997). His work mathematically described the risk as a function of the probability and consequences of failure, which is in turn a function of the expenditure in different mitigation areas (Equation 9-1). The risk is minimised using Lagrangian multipliers, subject to a constraint that the mitigation expenditure cannot exceed some allowable maximum (Equation 9-2).

$$R = \sum_{i=1}^n C_i(a_1 \dots a_m) P_i(a_1 \dots a_m) \quad (9-1)$$

$$\sum_{j=1}^m a_j = E_T \quad (9-2)$$

where R is the level of risk, C_i is the consequences of hazard i , P_i is the probability of hazard i occurring, a_j is the amount of expenditure on mitigation j , and E_T is the maximum allowable total expenditure.

Amende (1996) developed a numerical procedure for this optimisation for use on personal computers. It is Amende's initial numerical model that the work described here extends from.

The optimisation method used in this program breaks the total expenditure up into discrete steps, and looks at the incremental benefit of spending each successive step in any one mitigation area. The area that gives the best rate of return for that expenditure receives the additional funds. This process is described graphically in Figure 9-1, where there are two possible mitigation options (A and B), and the function that is to be minimised is R.

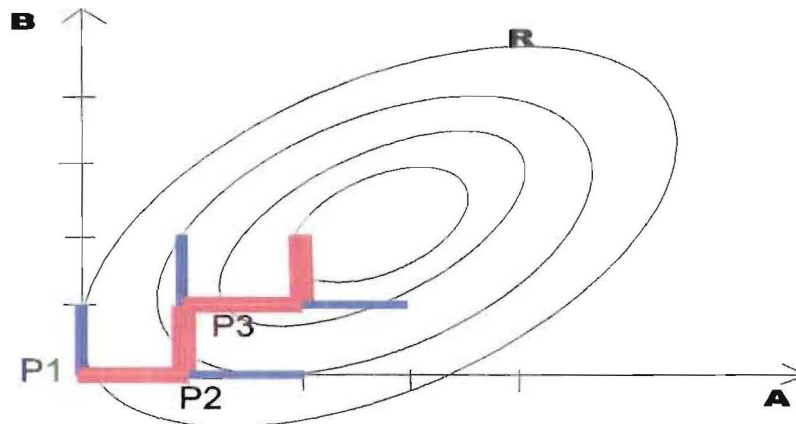


Figure 9-1: Graphical example of the optimisation method

Starting at point P1 on Figure 9-1, where the expenditure is zero (0, 0), the first step of expenditure (S) may be spent on either mitigation A (S, 0), or on mitigation B (0, S). As the objective is to minimise the value of R, the option chosen will be that which yields the minimum value of R. In Figure 9-1, spending the first step of expenditure on mitigation option A is optimum (point P2). In deciding how to spend the second step of expenditure, the risk is again evaluated for both options. The expenditure step is first spent on mitigation A (2S, 0) and secondly, on mitigation option B (S, S). The option that the money is spent on will be the one that gives the optimum reduction in R [in this case (S, S), point P3]. This process is repeated until the total allowable expenditure is spent.

The primary limitation of this search optimisation method is that it will only ever find the local maximum or minimum of a function (Aldby, 1974). To accommodate this, where possible (for some problems this will not be possible), the problem should be approached so that the function to be optimised is a uni-modal function, with only one maximum or minimum (whichever is of interest). This then means that the local maximum or minimum found will also be the global maximum or minimum.

9.2.2 Calculation Details

Existing Risk

The program initially establishes what the existing total risk is with no expenditure. This is calculated using Equation 9-3.

$$R_0 = \sum_{i=1}^n C_i P_i \quad (9-3)$$

C_i and P_i are the present costs and probabilities of the various possible failure modes. In the context of the Desert Road, the probability of failure is the frequency of road closure, and the consequences of failure would be the cost of each road closure. The costs and probabilities of failure may be reduced by investment in various mitigation options (A and B, etc).

Risk Reduction

In general, money spent to reduce risk has a diminishing rate of return. Initial amounts of money spent are very effective at reducing risk, but as the total expenditure increases, the effectiveness of each unit of expenditure is reduced. This diminishing rate of return means that the reduction in the probability or cost of failure takes on an exponential shape. In the program, this relationship is represented by a curtailed power, limited exponential relationship or a step function depending on the nature of the risk mitigation action.

The limited exponential relationship (Figure 9-2) reduces the probability or cost of failure exponentially. This relationship however recognises that in many situations the probability or cost can never be totally eliminated, imposing a limit to the amount of reduction possible. Most risks would fall into this category. It is possible for example to spend a large amount to improve the structural safety of a bridge, but there is always the possibility of an earthquake larger than the design earthquake.

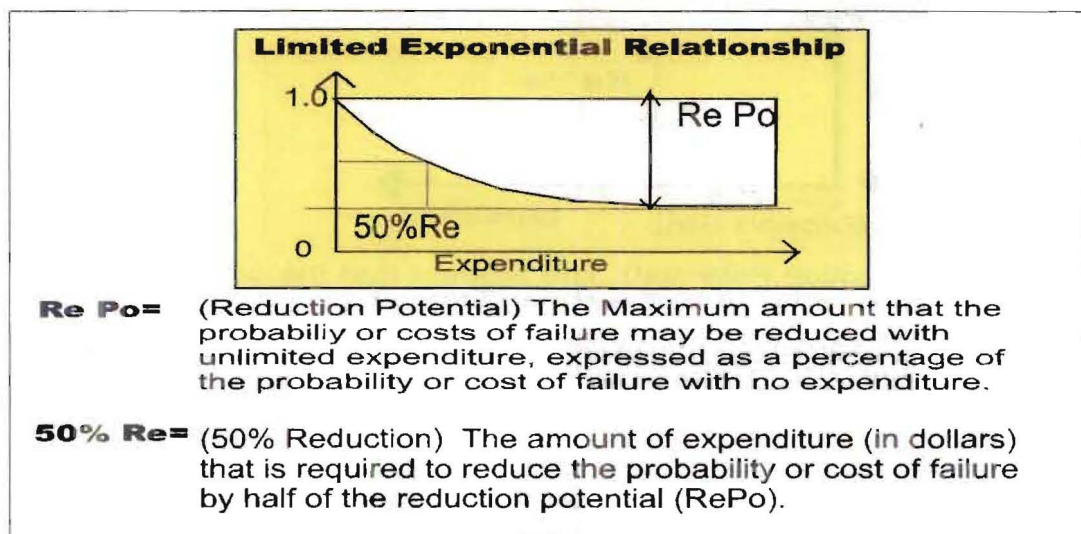


Figure 9-2: Limited exponential relationship

For each mitigation that has a limited exponential relationship with a particular hazard, the parameters to be input to the program are the reduction potential (Re Po) and the expenditure required to attain half of this possible reduction (50% Re).

The curtailed power relationship (Figure 9-3) reduces the probability or costs of failure to zero. Initial reductions in risk follow an exponential shape, but unlike an exponential curve, the curtailed power relationship achieves zero probability or cost. The parameters required for this relationship are the amount of expenditure required to reduce the cost or the probability of failure to zero (Max. Ex), and a shape factor. The shape factor ranges from zero to one, dictating the initial rapidity of the reduction in risk.

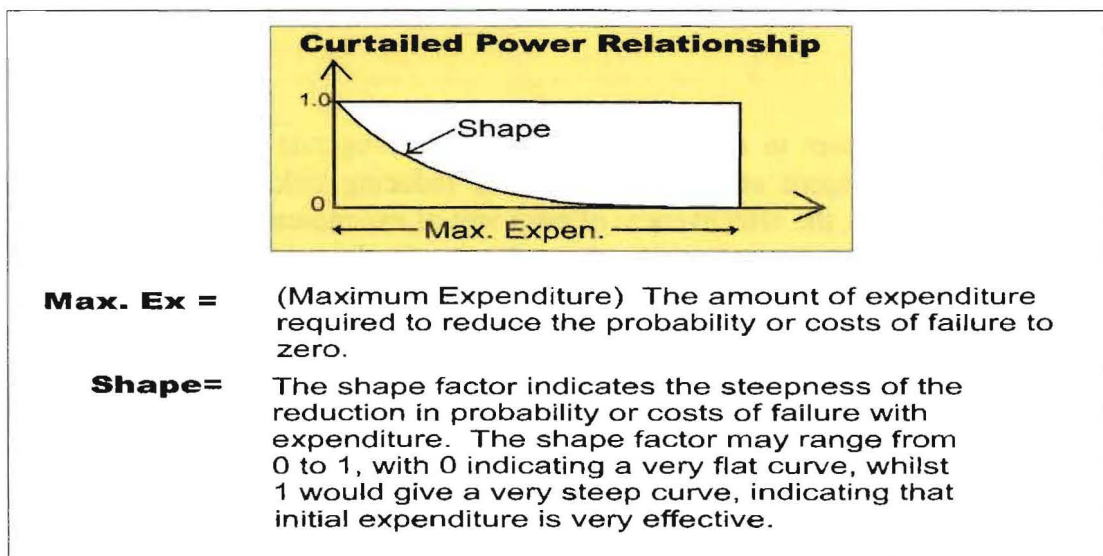


Figure 9-3: Curtailed power relationship

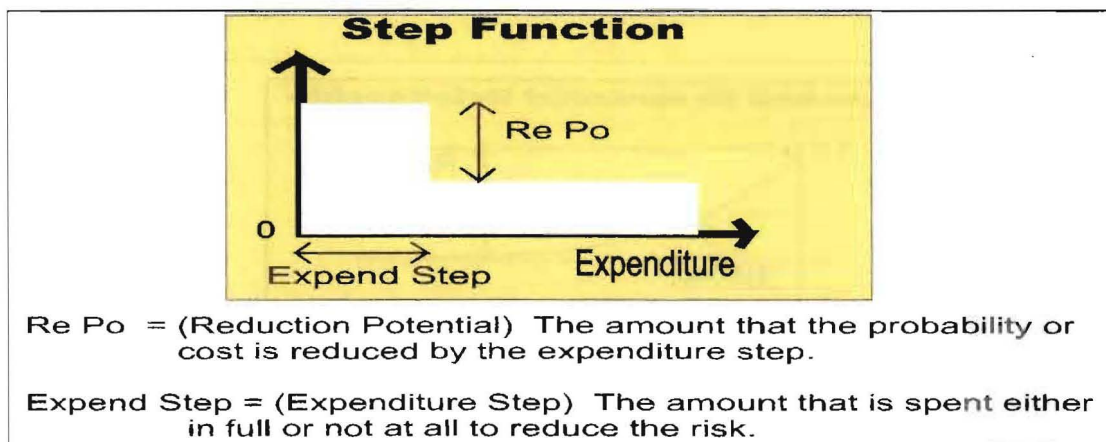


Figure 9-4: Step function

The third relationship available in the program to describe a reduction in risk through mitigation, is a discontinuous function called a step function (Figure 9-4). In this function, expenditure has no effectiveness until a threshold expenditure is reached. At this threshold, the probability or cost of failure is reduced by the reduction potential (Re Po). Any expenditure above this threshold then has no further effectiveness. This type of function is useful for representing scenarios such

as a bridge replacement, where the cost of the bridge is either met in full, or the bridge is not built at all.

Selection of the appropriate risk mitigation function is based on the nature of the mitigation action and the relationship between the reduction in risk probability and expenditure on mitigation.

Risk Optimisation

Expenditure on any of the mitigation options may reduce the cost (C) or probability (P) of one or more failure modes (Equations 9-4 and 9-5). The amount of this reduction is dictated by the risk reduction relationship.

$$C_i = C_i(a_1, \dots, a_m) \quad (9-4)$$

$$P_i = P_i(a_1, \dots, a_m) \quad (9-5)$$

where a_1 to a_m are the levels of expenditure on mitigation options 1 to m.

The optimal way to spend the first expenditure step is found by consecutively spending the money in each of the possible mitigation options (for example, A and B). For each spending regime, the new costs and probabilities of each of the failure modes are found, and the total risk is calculated (Equations 9-6 and 9-7).

$$R_{1a} = \sum_{i=1}^n C_i(A) P_i(A) \quad (9-6)$$

$$R_{1b} = \sum_{i=1}^n C_i(B) P_i(B) \quad (9-7)$$

The resulting total risks (R_{1a} and R_{1b}) are compared, and the spending regime that results in the minimum risk is selected. Supposing that it is optimal to spend the first step of expenditure on option A, it is next determined how to spend additional steps of expenditure (Figure 9-5).

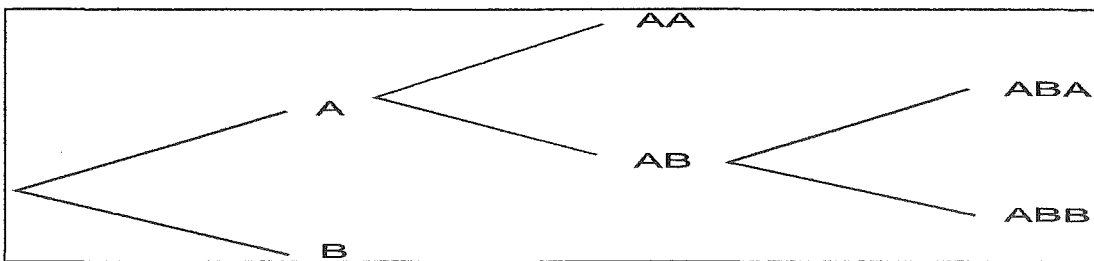


Figure 9-5: Decision tree for optimum spending distribution

In the second round of calculations, the expenditure step is again consecutively spent on each of the options (Equations 9-8 and 9-9).

$$R_{2a} = \sum_{i=1}^n C_i(AA) P_i(AA) \quad (9-8)$$

$$R_{2b} = \sum_{i=1}^n C_i(AB) P_i(AB) \quad (9-9)$$

The spending regime to give the minimum total risk is how the second expenditure step should be spent. This process is repeated until the expenditure spent equals the total expenditure available.

9.2.3 Incorporating Step Functions

It was mentioned earlier that the optimisation method used here is only able to locate local minimums and maximums. It is thus necessary, where possible, to approach the problem to be analysed so that the risk function to be minimised is uni-modal.

There are some mitigation projects however that have a discontinuous effect on the probability or cost of failure. Spending below a certain level has no effect upon an existing risk, whilst at or above some threshold expenditure, the probability or cost of the failure mode is reduced by a set amount. If this is the cause of a problems multi-modal nature, then this can be removed by specifying the discontinuity as a “step function”. The program deals with step functions differently than the other continuous risk functions.

To keep the risk function uni-modal, the program essentially optimises the same problem twice. The first time it assumes that no money is spent on the step option, effectively removing the step function from the problem to be solved. The second time, the program assumes that the expenditure threshold of the step function is spent in whole. It then optimises how to best spend any remaining expenditure. The spending regime that minimises the total risk is chosen.

Going back to the example earlier of two mitigation modes A and B, let us this time assume that mitigation mode B is a step function. Either an amount B_{step} is spent, or no money is spent on mitigation option B at all. This means that the risks for the various options for spending the money become:

- 1) assuming that no money is spent on option B, summing the total risk over all the hazards

$$R_1 = \sum_{i=1}^n C_i(A) P_i(A) \quad (9-10)$$

- 2) assuming that the threshold expenditure B_{Step} is spent on option B

- a) if the expenditure is less than B_{Step}

$$R_2 = \sum_{i=1}^n C_i P_i \quad (9-11)$$

- b) if the expenditure is equal to B_{Step}

$$R_3 = \sum_{i=1}^n C_i(B_{\text{Step}}) P_i(B_{\text{Step}}) \quad (9-12)$$

- c) if the expenditure is greater than B_{Step} (where $A = \text{expenditure} - B_{\text{Step}}$)

$$R_4 = \sum_{i=1}^n C_i(AB_{\text{Step}}) P_i(AB_{\text{Step}}) \quad (9-13)$$

Each of the applicable risks are calculated, with the lowest risk determining whether or not it is optimum to invest in option B.

9.2.4 Mitigation Combinations

A further degree of complexity that may exist in many problems to be analysed is interdependence between possible mitigation options. There may be cases where two mitigation options are mutually exclusive. Examples are the use of different chemicals that cannot be applied together, or engineering projects requiring different physical conditions. There may also be dependence between mitigation modes, where one project needs to be completed before the other can become feasible.

These interdependencies between the mitigation options mean that there will be several possible mitigation combinations. To find the optimum combination, the program firstly identifies all of the possible combinations, optimises each one in turn, and then compares each resulting risk to find the optimum combination.

To identify all of the possible mitigation combinations, the program makes use of binary numbers. The total possible number of combinations will be 2^N , where N is the number of mitigation options. The computer then looks at the first 2^N binary numbers, and assesses whether or not the location of the 0's and 1's fit with the interdependence criteria of the mitigation options. For example, the number 5 in binary is 101, this means that expenditure would only be spent on the 1st and 3rd mitigation options. The program simulates only those combinations that meet these interdependence criteria. This process is illustrated in Table 9-1, where mitigation options A and B are mutually exclusive.

Table 9-1: Selection of mitigation combinations using binary numbers

Binary Number	Option A	Option B	Option C	Acceptance
0	0	0	0	*
1	0	0	1	*
2	0	1	0	*
3	0	1	1	√
4	1	0	0	*
5	1	0	1	√
6	1	1	0	×
7	1	1	1	×

In this example, the combinations associated with binary numbers 6 and 7 are unacceptable (indicated by a cross in the final column), as both have mitigation options A and B occurring together. Binary numbers 3 and 4 are acceptable combinations (indicated by the tick in the final column), with neither resulting in A and B coinciding. The remaining binary numbers, with an asterisk in the acceptance column, are not simulated. Although meeting the interdependence criteria, these combinations are simply subsets of the two combinations that are to be simulated, but leaving out a mitigation option. If it is optimum for no resources to be spent on this mitigation option, then that would be found in the optimisation of the more complete combination. Cutting out these subset combinations reduces the number of combinations to be simulated, and thus the processing time.

9.2.5 Program/User Interface

The program is run in Excel, with all the calculations and program sequence being automated using macros within the Excel file. The user is guided through the program with detailed instructions, and buttons to move to the next stage of the program. There are also help buttons to provide additional explanations as the program is run.

The program is activated by hitting a start button, that clears all existing data and requests the user to specify both the number and names of the failure and mitigation possibilities. With this information, the program then draws up the matrices and tables that are required. The program uses a total of five Excel sheets for data input and results. The format of four of these sheets are shown in Appendix H. In the program two copies of the “risk reduction relationships” sheet are used. One sheet is used to specify the effectiveness of mitigation to reduce the hazard consequences, and one for its effectiveness in reducing the probability of a negative outcome.

The first screen in the program allows the user to specify the accuracy with which the optimisation is to be run (governed by the step size) and the total funds available for mitigation. The program then moves to the interdependence sheet, where the user is requested to complete the interdependence matrix, specifying the relationships between the various mitigation options (Section 9.2.4). The program next moves to a table, where the user defines the relationships between the mitigation options and the costs of the various failure modes (Section 9.2.2). Once the relationship types are specified, the parameters of the relationships are input. The following screen (not shown in Appendix H) shows a similar table, this time defining the relationships between the mitigation options and the probabilities of the different failure modes occurring.

With all of the relevant information input, the program asks the user if they are ready to optimise the model. If the answer is “yes”, then the user can observe the program optimising the possible combinations, and finally showing the optimum spending portfolios.

9.2.6 Program Output

The form of the program output is a table showing the minimum level of risk attainable for a given amount of total expenditure. This allows the decision maker to calculate the worth of spending additional funds, as the effectiveness of the spending diminishes. The program also shows the distribution of spending amongst the possible mitigation modes that give the optimum level of risk. This is especially effective at showing the changing combinations that are optimum at different spending levels.

Table 9-2 is an example of such an output table. As the total expenditure is increased to 1000 units, the total risk is reduced by 60 percent. Half of this reduction however is attained by spending only 300 units, indicating the significant diminishing rate of return of the expenditure. A decision maker can use this information to decide how much expenditure it is worth spending to reduce the total risk.

Table 9-2: An example of the program output showing the optimum risk and expenditure distribution to achieve the optimum risk at each expenditure level

Expend.	Risk	Option 1	Option 2	Option 3	Option 4
0	1647	0	0	0	0
50	1535	0	50	0	0
100	1432	50	50	0	0
150	1346	80	70	0	0
200	1273	120	80	0	0
250	1212	150	100	0	0
300	1160	190	110	0	0
350	1116	220	130	0	0
400	1030	0	100	300	0
450	958	0	150	300	0
500	903	0	200	300	0
550	861	0	250	300	0
600	827	0	270	300	30
650	798	0	300	300	50
700	772	0	320	300	80
750	749	0	340	300	110
800	729	0	370	300	130
850	712	0	390	300	160
900	696	0	410	300	190
950	682	0	430	300	220
1000	670	0	460	300	240

Table 9-2 also clearly shows the changes in the optimal mitigation combination, with increasing spending. Option 3 is a discontinuous function, with an expenditure step of 300 units. It is not until the total expenditure is up to 400 units that this option becomes worthwhile. Similarly, although option 1 is dominant when expenditures are small, it does not feature in expenditures greater than 400 units. Option 4 only becomes viable at high expenditure levels.

9.2.7 Incorporating Uncertainty

As the program interface uses Excel spreadsheets, it is possible to incorporate @RISK capabilities into the optimisation process. Instead of entering parameters as set values, @RISK allows the user to specify parameters as a distribution of the values that the parameter can take. For example a mean and standard deviation for a parameter that is normally distributed. @RISK then randomly generates a value for each parameter from their respective distributions. These values are then used to find the optimum spending regime for that combination of parameters. The process is repeated many times, building up a profile of how the optimum spending regime alters with changes to the program input parameters. This process is called Monte Carlo simulation, and is a valuable tool for gauging the sensitivity of model results to any uncertainty in the input parameters.

Existing models that have been developed using Excel can be directly referenced for the parameters required for the optimisation input. This eliminates the need to characterise parameters, such as the probability or consequences of a hazard, as a

single probability distribution. Instead, the input parameters can be calculated as the combination of several factors.

When examining the range of spending combinations that are optimal for a given spending regime, it is possible to identify projects that are definitely worthwhile, attaining a consistent level of expenditure. The value of other projects may be quite fickle, being worthy of large amounts of expenditure in some instances, but in others not obtaining any funding. It is possible with @RISK to identify those parameters to which the worth of a project is most sensitive. If a project is then proceeded with, effort can be put into ensuring that these important parameters remain near their optimum level.

9.2.8 Future Developments

This risk optimisation software is still in its initial stage of development, and as such there are some important aspects that still need further consideration. One such area is the addition of increasing risk functions. Some mitigation options, while reducing one type of risk, may actually increase the risk of another hazard. An example could be the application of salt onto the road surface to reduce the risk of closure due to ice. In some extreme instances, this salt could migrate through the concrete structures of bridges along the road, and cause serious corrosion of the reinforcing bars inside. This will jeopardise the structural strength of the bridge, and may increase its vulnerability to failure during an earthquake. At present, this mode of interdependency is not included in the model. Problems are likely to occur when trying to implement this feature into the model however, as the total risk function may lose its uni-modal characteristics.

Optimisation techniques such as annealing (Kirkpatrick *et al.*, 1983) may be an option to overcome this uni-modality requirement of the model. Annealing basically introduces a random component into the optimisation process, where the step that attains maximum risk reduction is not always selected. There is a probability that a non-optimal expenditure step will be chosen, effectively allowing the optimisation to “jump over” a local maximum or minimum, from one valley or peak to another valley or peak.

Another aspect that currently limits the utility of the risk optimisation software is its requirement that successive reductions in risk by different mitigation options be linear. The model assumes that the mitigation options have the ability to reduce the risk of a particular hazard by a certain percentage with a given expenditure. If two mitigation options are used that both reduce the risk of a particular hazard, then the model assumes that the total reduction in risk will be:

$$\text{Final Risk} = \text{Original Risk} * (1 - \text{Proportion of risk reduced by mitigation A}) * (1 - \text{Proportion of risk reduced by mitigation B}) \quad (9-14)$$

This does not account for any redundancy or interdependency between the mitigation options. If one mitigation option eliminates one consequence of the hazard from occurring, then the second mitigation will have no further effectiveness in reducing this consequence (though it may still work to reduce other consequences). This

means that the model will overestimate the effectiveness of using these two mitigation option in combination.

9.3 Potential Applications of Methodology

During initial development of this risk optimisation software, it was envisaged that this would provide a tool that could be used in determining the optimum portfolio of mitigation options that should be used on the Desert Road. Unfortunately however, the complexity of the Desert Road problem means that at this stage of development, the risk optimisation software does not lend itself to the type of analysis required.

The consequences of the road closure depend upon the duration of closure of the Desert Road, and of the alternative routes. When determining the effectiveness of mitigation options, the effectiveness is often in terms of shorter road closure durations on either the Desert Road, its alternative routes, or both. The reduction in cost attainable with a mitigation will not be directly proportional to the reduction in closure time on any one route. This means that each hazard model must be re-simulated to find the actual risk reduction attained, and thus the effectiveness of the mitigation.

There is also the significant problem of non-linearity in the risk reduction attained when two or more mitigation options are used together. Many of the mitigation options considered in this thesis (e.g. CMA, RWIS, and grader equipment) are intended to be used together. As the program does not deal with non-linearity of the effectiveness of mitigation options, the results obtained would not be accurate.

Although the risk-optimisation software seems not to be suited for interpreting highly detailed, non-linear problems, it is anticipated that it will still be of great value for preliminary evaluations. Often it is possible to get “gut feeling” estimates of how much a mitigation may cost, and how effective it will be. By running the software with initial ball-park estimates of parameters, in conjunction with @RISK to include uncertainties, it is possible to develop a “feel” for the problem. The parameters that are likely to be significant can be identified, and by simulating the optimisation a few times, the results should indicate the general type of spending regimes that are likely to be more effective (i.e. spend all the resources on one project, or to distribute it more evenly). This can be especially useful where there are numerous possible expenditure options, many of which may be mutually exclusive or dependent.

9.4 Conclusions

The risk optimisation program provides a very useful tool for decision makers faced with complex problems. It is especially useful for preliminary evaluations of problems, to help decision makers get a “feel” for the problem, and identify the key parameters. The risk balancing program accounts for interdependence between

mitigation options, and provides a format for the effectiveness of each mitigation option to be specified. The incorporation of uncertainty into the analysis is also very valuable, as uncertainty adds significantly to the complexity of any problem. It should be emphasised that although the program provides a tool that will predict the optimum distribution of resources, the quality of the predictions, and thus the decision made, will always be limited by the quality of the source data.

9.5 References

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10. CONCLUSIONS

10.1 Use of Risk Assessment Methodology in Road Network Evaluation

Risk is a function of both the probability and consequences of an undesirable event occurring. Many of the hazards that threaten the integrity of our roading network are rare events. Since they are not everyday occurrences, it can be difficult to determine their actual probabilities of occurrence and their consequences. Uncertainty surrounding these two factors can make it unclear what is the actual level of risk, and how much it is worth spending to try to alleviate this risk.

Probabilistic risk assessment requires that all uncertainty about parameters, such as uncertainty surrounding the probability or consequences of an event, be included in an analysis. Uncertainty regarding these parameters will propagate through into uncertainty regarding the total risk, or benefit-cost ratio for any mitigation. Probabilistic risk assessment methods ensure that this uncertainty is shown explicitly to the decision maker using a probability distribution for the benefit-cost ratio. This provides far more information to the decision maker than does the traditional point estimate of the benefit-cost ratio. Where two risk mitigation options have a similar average benefit-cost ratio, one option may be subject to more uncertainty. Expressing the benefit-cost ratio information in terms of probabilities, allows the decision maker to see the likelihood of various benefit-cost ratio targets being met.

Particularly for complex assessments, where the actual uncertainty of some parameters may not be intuitive, or interaction and compounding of uncertainties occur, it is vital for the actual uncertainty about a prediction to be made clear to the decision maker. Probabilistic risk assessment methodology, using Monte-Carlo simulation, provides a transparent tool with which to do this.

As well as illustrating the total effect of uncertainties on the final results, probabilistic risk assessment also allows the sensitivity of the model results to the uncertainty of individual parameters to be easily found. Sensitivity analysis will indicate parameters upon which the model result is especially dependent. Efforts to reduce the amount of uncertainty in the model predictions can then be concentrated on these parameters. If a positive result is especially dependent upon one or two parameters exceeding certain thresholds, then attempts can also be made to ensure that this occurs.

Sensitivity analysis is important in another aspect as a check that the predictive models are behaving in a way that would be expected. If the model proves to be sensitive to an unexpected parameter, then this may indicate that there are feedback loops occurring in the model, increasing the importance of this parameter. It may also indicate that there are some design faults or limitations in the predictive model that need to be improved.

A major barrier to the use of probabilistic risk assessment methodology is the increased amount of time and effort that is required to perform such an analysis. It does require more work, in that effort is required to find the probability distribution of values that a parameter may take, as opposed to a point estimate. For complex projects, where there is a great deal of uncertainty, this additional effort may be substantial. It is for these projects however that there is the greatest need for uncertainty analysis, and the extra effort is likely to be justified. There are few major infra-structure projects where expenditure on a risk assessment would not be cost effective.

The analysis of closure risks that was done here was quite detailed, and did require considerable effort. Given that, now that an initial assessment has been done, it is anticipated that any subsequent analysis would be much easier. It is likely to be easier in the sense that this pilot study indicates the areas to which the analysis results are most sensitive. Any future study may then concentrate modelling efforts on these factors, and simplify the analysis for other areas that proved to be less important. This study also provides “ballpark” estimates of the magnitude of existing risks, that can be used as a benchmark in the preliminary evaluation of whether or not a detailed analysis of some mitigation is warranted. If a similar analysis of natural hazard risk were to be done, this study will also provide a starting point for the approach that could be taken. Any analysis done after this, will at least have an indication of sources and availability of information, and the type of approach that can be used or developed upon.

The risk optimisation software described in Chapter 9 also shows promise as a decision tool that can be used in preliminary assessment of risk mitigation. Although it was not directly used in this study, the software would be very useful for enabling an analyst to get a “feel” for the problem at hand. Using initial ballpark estimates for parameters, it is possible to see which are the important parameters, and the type of spending portfolios that are likely to be optimum. Further development of this software should lead to its improved utility.

10.2 Cost of Road Closures to the New Zealand economy

This study has looked at the risk of closure of the Desert Road, not only in terms of its disruption to travellers using the Desert Road, but also to all users of the Central North Island road network. The system wide impact of road closures includes the cost of additional travel along the alternative routes, the cost of increased accident exposure, and the lost user benefit of those trips that are cancelled because of the road closure. These were predicted using a traffic network model that was developed for the whole of the central part of the North Island. It is estimated that closure of the Desert Road costs the New Zealand economy approximately \$8,000 per hour of closure. These are only the tangible costs of road closure to the road user. There will be other effects of the road closures, particularly social impacts associated with community isolation and shifting of business revenue.

Assessing the system wide effects of road closure has the advantage of also being able to find the consequences of more than one road link being closed, such as the Desert Road and one of its alternative routes. Costs of closure inflate when more than one road link is closed, but are particularly high when both the north-south links across the Central Plateau are cut. When both the Desert Road and State Highway 4 (on the other side of Mount Ruapehu) are closed, north-south drivers must divert either via New Plymouth or Napier. These longer diversions cost the New Zealand economy nearly \$23,000 per hour that the roads are closed.

Some hazards will have a wider geographic impact than others. For example, a volcanic eruption will spread ash over a wide area, affecting transport services on many road links. Closure of the Desert Road due to a volcanic eruption is therefore likely to be accompanied by simultaneous closure of many other road links. This will limit diversion options for travellers, increasing the cost of the Desert Road closure. In this study, the vulnerability of the alternative routes to simultaneous closure with the Desert Road was assessed, acknowledging the differing consequences of a hazard, depending on its geographic impact.

Results indicated that snow and ice is the most significant hazard to the Desert Road, costing the New Zealand economy an average of \$1.9 million a year. There is a large variability of this annual cost predicted by the model, reflecting the variability of closure due to snow and ice. Even with this variability though, snow and ice still dominates over the other hazards as the major closure mechanism. For this hazard cost effective mitigation options can be implemented.

Earthquakes also impose a relatively large risk to the Desert Road, costing the New Zealand economy an average of \$1.5 million a year. Traffic accidents and volcanic events pose much less threat, each costing the country on average 0.3 and 0.2 million dollars a year respectively.

10.3 Mitigation Strategies for the Desert Road

This study assessed the economic impact of road closures, and the economic worth of some potential mitigation options. There was no attempt, other than to highlight areas that may be of concern, to quantify or judge the importance of the social or environmental effects of the mitigation options. As the economic viability of any mitigation is only part of the information upon which a decision is based, these will need to be investigated further before any course of action is decided on.

The aim of this study was to investigate the usefulness of the risk assessment methodology as a tool in road network evaluation. The evaluation of mitigation options was done to illustrate an approach that could be used, and the type of results expected. Although the analysis was as thorough as possible, it is not expected that the analysis done here would be used, as is, for any final project evaluation. The appropriateness of the model assumptions and simplifications would need to be examined within the context of the evaluation.

Snow and ice pose the greatest risk to the Desert Road. Potential mitigation options to alleviate this risk include the application of chemicals to the road surface, either with or without road weather information system (RWIS) support. Salt and Calcium Magnesium Acetate (CMA) were investigated as both de-icing and anti-icing agents. Application of salt was the more economically beneficial option of the two, though the environmental effects of this chemical are likely to counter this dominance. CMA has far fewer environmental effects than salt when applied to the road surface, but is very expensive to supply. Although not as high as that for salt, the benefit-cost ratio for CMA application indicates that this is still a very cost effective solution. Using CMA in conjunction with RWIS has many advantages over using CMA on its own. Application of CMA in an anti-icing capacity rather than as a reactionary measure, should prevent ice from forming on the road surface, and thus prevent ice related accidents. Less CMA will also need to be spread when used with RWIS, reducing the supply costs of CMA significantly. These savings in CMA supply costs actually exceed those to set up and run the RWIS, meaning that it is cheaper to use CMA and RWIS together than to use CMA on its own. The benefit-cost ratios predicted for CMA and RWIS range from 14 to 60 (the 5th and 95th percentiles), indicating that this option is definitely worthy of further investigation.

Earthquakes are the second largest threat to the integrity of the Desert Road. As damaged bridges can take a long time to repair, it could be expected that temporary access, using Bailey bridges or fords, would be put in place to restore traffic flow whilst repairs proceed. Fords are the cheaper option for establishing temporary access, though the location of the Desert Road in a National Park is likely to prevent these from being constructed. Because of the high capital cost of constructing a Bailey bridge, there will be an optimum threshold closure duration above which it is worth constructing a Bailey bridge. This optimum threshold for the Desert Road was found to be 70 days. Constructing Bailey bridges after an earthquake, for access past any bridge where the closure duration is expected to be longer than 70 days, has a benefit-cost ratio of 150.

The Waihohunu bridge, on the Desert Road, has concrete column piers that were designed prior to 1973. This means that these columns may have non-ductile detailing, and be prone to catastrophic failure in an earthquake. Retrofit of this bridge using steel jackets, costing around \$22,000, would reduce the vulnerability of this bridge to failure or damage in an earthquake. This would also reduce the number of Bailey bridges that would need to be constructed in the aftermath of an earthquake. The capital cost savings expected through fewer Bailey bridges being required is expected to exceed the cost of retrofit. This means that there is a negative capital cost expected for Transit New Zealand to retrofit this bridge. This retrofit was awarded a benefit-cost ratio of 99 to reflect its economic value.

As the vulnerability assessments of the Waihohonu and other bridges in the central North Island were only preliminary estimates, this high benefit-cost assessment may not be a true reflection of the worth of actually doing the retrofit. It does however indicate the worth of further investigating the actual vulnerability of this bridge.

The economic analysis of these mitigation options indicate that the costs of traffic disruption, and the closure of strategic road links due to natural hazards, are

significant. These costs are often not fully quantified or taken into account in traditional project evaluation. The analysis done here provides a method that can be used to quantify traffic disruption due to natural hazards, and gives estimates of the magnitude of these expected costs.

APPENDIX A

Mean Intervals between Volcanic Eruptions

Mt Ruapehu		
Erupted Volume	Return Period	Period Calculated From
$\geq 10^4 \text{ m}^3$	0.12 yrs (44 days)	Extrapolated
$\geq 10^5 \text{ m}^3$	1.6 yrs	1864-1984
$\geq 10^6 \text{ m}^3$	23 ± 11 yrs	1864-1984
$\geq 10^7 \text{ m}^3$	320 yrs	1800y B.P.
$\geq 10^8 \text{ m}^3$	4500 yrs	1800 yrs B.P.
$\geq 10^9 \text{ m}^3$	63000 ± 31000 yrs	200,000 yrs B.P.
$\geq 10^{10} \text{ m}^3$	900 000 yrs (max)	Extrapolated

Mt Ngauruhoe		
Erupted Volume	Return Period	Period Calculated From
$\geq 10^4 \text{ m}^3$	0.44 yrs (161 days)	1839-1948
$\geq 10^5 \text{ m}^3$	1.9 ± 2.1 yrs	1839-1984
$\geq 10^6 \text{ m}^3$	26 ± 36 yrs	1839-1984
$\geq 10^7 \text{ m}^3$	53 ± 45 yrs	1839-1984
$\geq 10^8 \text{ m}^3$	560 yrs	Extrapolated
$\geq 10^9 \text{ m}^3$	3300 yrs (max)	Extrapolated

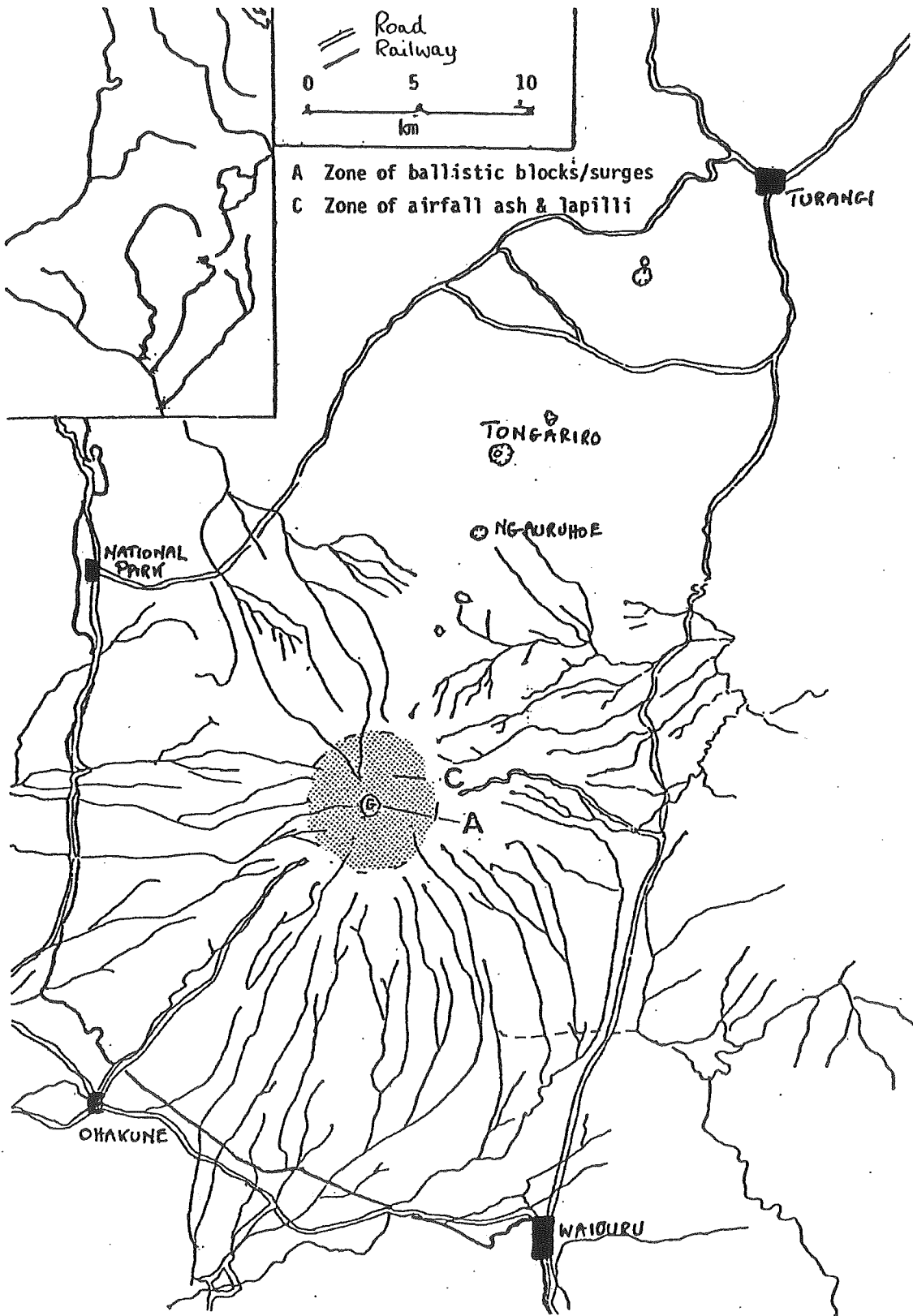
Tongariro ($\geq 10^8 \text{ m}^3$ includes Ngauruhoe eruptions)		
Erupted Volume	Return Period	Period Calculated From
$\geq 10^4 \text{ m}^3$	4 yrs	Extrapolated
$\geq 10^5 \text{ m}^3$	24 ± 14 yrs	1850-1984
$\geq 10^6 \text{ m}^3$	140 yrs	1850-1984
$\geq 10^7 \text{ m}^3$	820 yrs	1850-1984
$\geq 10^8 \text{ m}^3$	4800 ± 1000 yrs	12500 yrs B.P.
$\geq 10^9 \text{ m}^3$	28000 yrs	12500 yrs B.P.
$\geq 10^{10} \text{ m}^3$	160 000 yrs (max)	12500 yrs B.P.

Taupo		
Erupted Volume	Return Period	Period Calculated From
$\geq 10^4 \text{ m}^3$	270 yrs	Extrapolated
$\geq 10^5 \text{ m}^3$	490 yrs	Extrapolated
$\geq 10^6 \text{ m}^3$	870 yrs	Extrapolated
$\geq 10^7 \text{ m}^3$	1600 yrs	Extrapolated
$\geq 10^8 \text{ m}^3$	2800 yrs	Extrapolated
$\geq 10^9 \text{ m}^3$	4900 yrs	40,000 yrs B.P.
$\geq 10^{10} \text{ m}^3$	8800 yrs	40,000 yrs B.P.
$\geq 10^{11} \text{ m}^3$	60,000 yrs	320,000 yrs B.P.

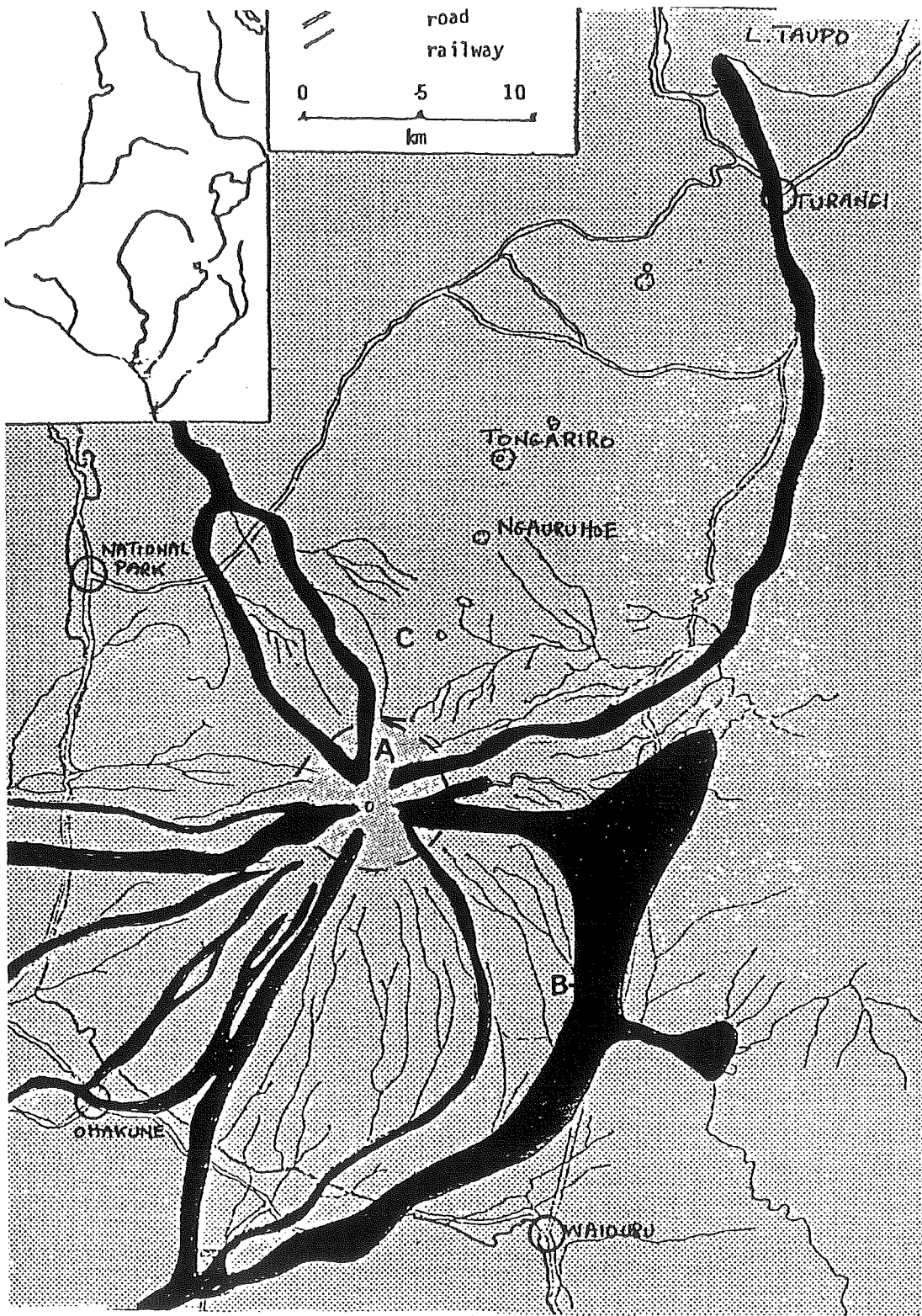
(Interval periods derived from Latter, 1985)

APPENDIX B

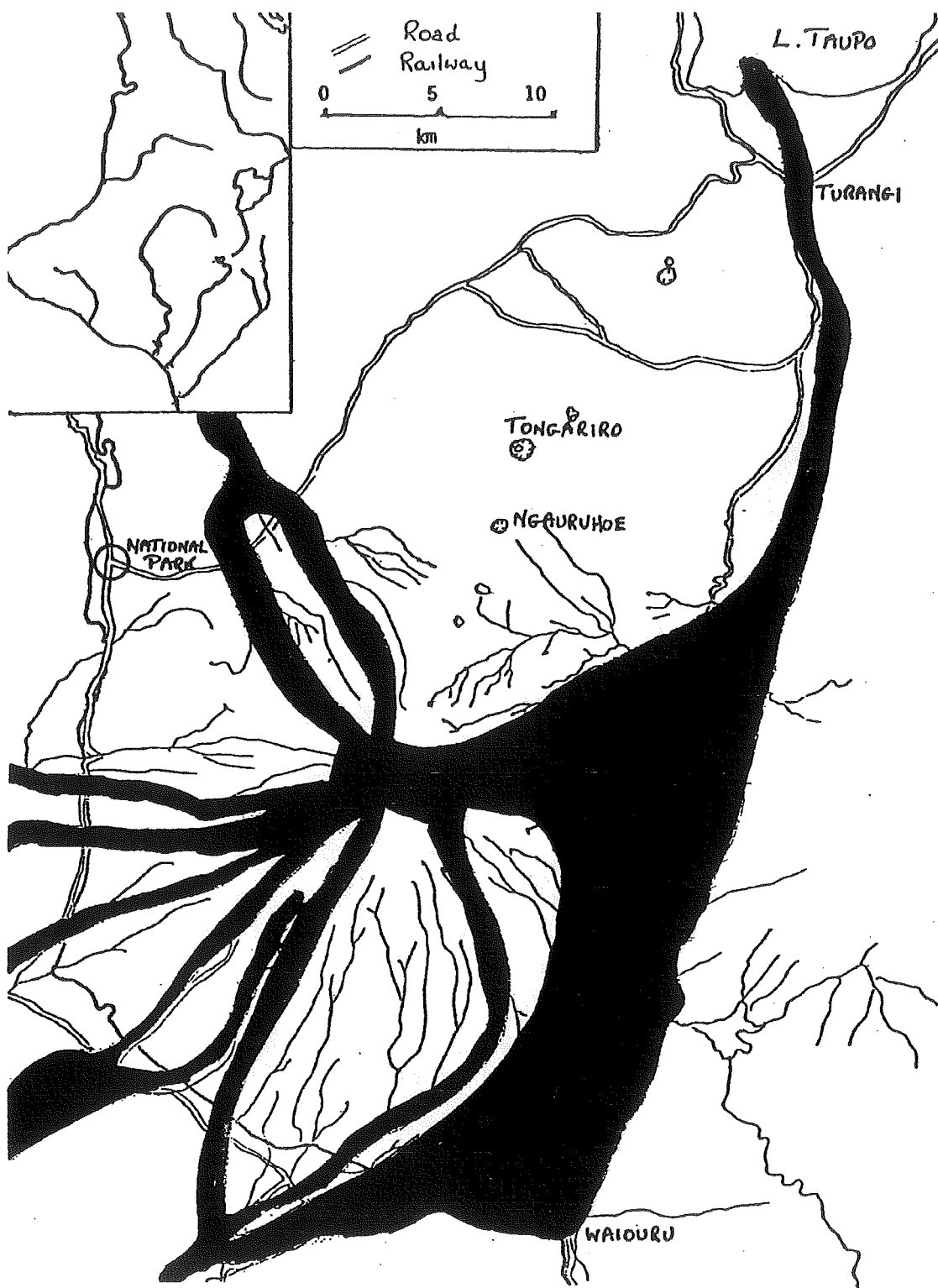
Hazard Zone Maps for Mt Ruapehu



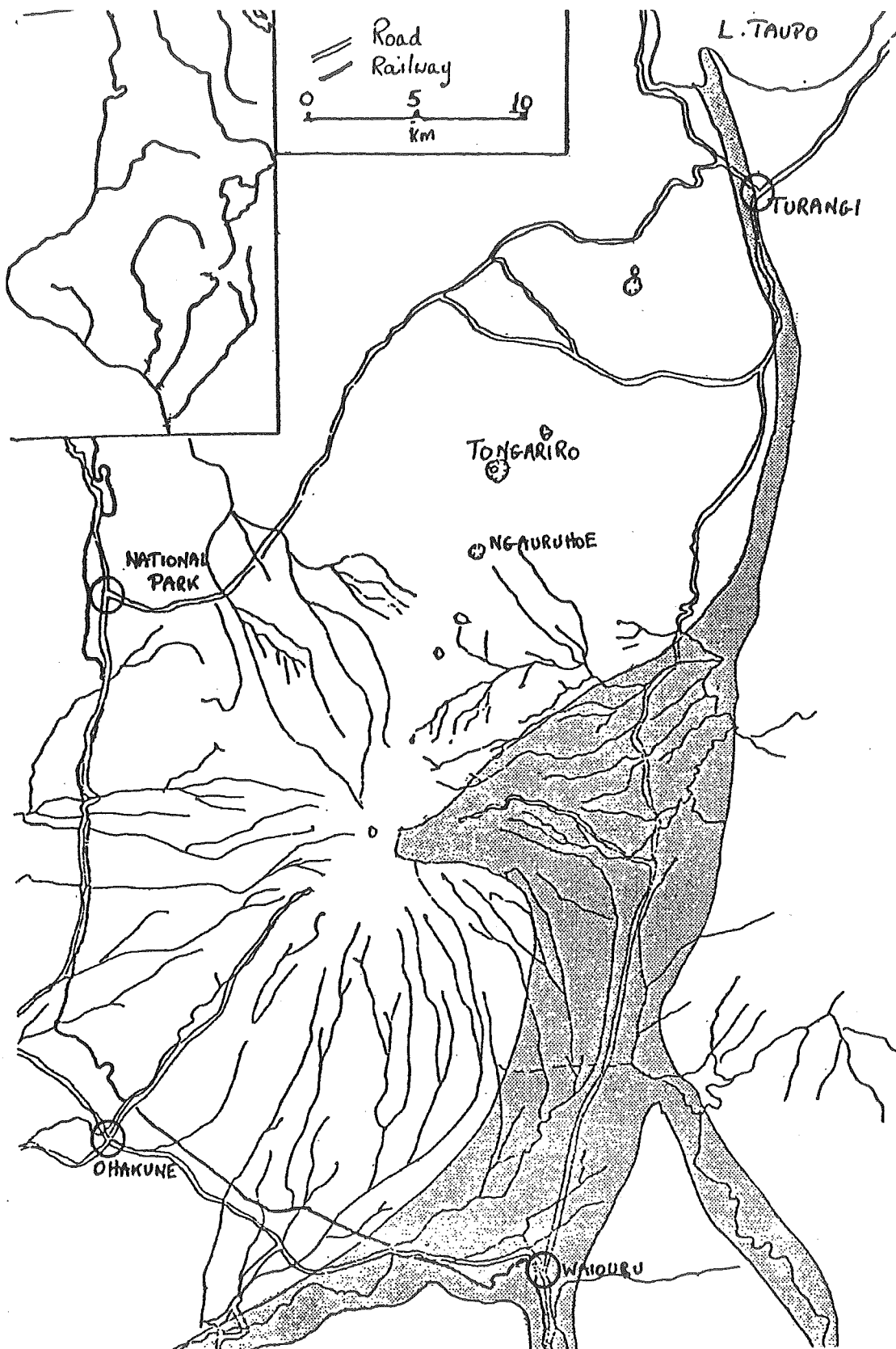
Zones of volcanic risk associated with phreatomagmatic eruptions of Ruapehu of about 1 year frequency.



Zones of risk from a phreatomagmatic eruption of frequency = approx. 100 years. Zone types as for previous figure.



Risk zones associated with lahars generated by a major phreatomagmatic eruption of return period of several hundred years. Whole area would also be covered by ash and lapilli.



Zone of risk associated with lahars produced by collapse if the southern wall of Crater Lake Basin.

APPENDIX C

Closure durations along Alternate Routes due to Lahars formed at Ruapehu

Approximate Return Period (Years)	Associated Lahar Size (m ³)	Duration of Road Closure (Days)			
		SH1 (Desert Road)	SH49 (Ohakune - Waiouru)	SH4 (Turangi - Nat. Park)	SH41 (Manunui - Turangi)
1 - 3 years	10 ³	0	0	0	0
10 - 30 years	10 ⁵	0	0	0	0
100 years	10 ⁶	20-40	5-10	0-1	0
Several Hundred Years	10 ⁷	120-180	60-90	0-5	15-30
Crater Wall Failure	10 ⁹	120-180	90-120	0	0

Estimated closure durations of alternative routes following lahar events. Derived from estimating the extent of damage to the road from the lahar paths shown in Appendix B.

APPENDIX D

Seismic Vulnerability Calculations for Desert Road Bridges

POUTU CANAL BRIDGE

Movement Joint Vulnerability Rating

Movement Joints in Structure? (Y/N)

Y

fseat

0.147478

fink

1

fskew

1.707097

f

0.25176

Vmj

2.517596

Support Overlap Length Known (Y/N)

Y

Overlap Length (mm)

745

Bridge Constructed Before 1973? (Y/N)

#####

Length of Bridge (m)

31.6

Horizontal Earthquake Linkage Bolts? (Y/N)

N

Skew Angle

22.5

PC Girder Bridge with Full Width Diaphragm Movement at Joints? (Y/N)

N

Column/Wall Vulnerability Rating

Is there More than 1 span? (Y/N)

N

Vcw

0

R/C Column Present? (Y/N)

#####

Designed Prior to 1973? (Y/N)

0

Foundation/Abutments Vulnerability Rating

Erosion/Scour Factor

1.5

feros

1.5

Vfa

4.5

Is there more than one span?

N

Is the Bridge length >5m

Y

Vulnerability Rating of the Structure

Soil Factor

1.5

Vstruc

4.5

Vulnerability	V
POUTU CANAL BRIDGE	5.1

PUKETARATA STREAM BRIDGE

Movement Joint Vulnerability Rating	fseat	fink	fskew	f	Vmj
Movement Joints in Structure? (Y/N)	0.295	1	1.999	0.591	5.9

Y

Support Overlap Length Known (Y/N)

Y

Overlap Length (mm)

714

Bridge Constructed Before 1973?

#####

Length of Bridge (m)

48.8

Horizontal Earthquake Linkage Bolts? (Y/N)

N

Skew Angle

44.2

PC Girder Bridge with Full Width Diaphragm Movement at Joints? (Y/N)

N

Column/Wall Vulnerability Rating

Vcw

Is there More than 1 span? (Y/N)

5

Y

R/C Column Present? (Y/N)

N

Designed Prior to 1973? (Y/N)

#####

Foundation/Abutments Vulnerability Rating

feros

Vfa

Erosion/Scour Factor

1.5

6

1.5

Is there more than one span?

Y

Is the Bridge length >5m

Y

Vulnerability Rating of the Structure

Vstruc

Soil Factor

6

1.5

Vulnerability	V
PUKETARATA STREAM BRIDGE	6.5

MANGATAWAI STREAM BRIDGE

Movement Joint Vulnerability Rating

Movement Joints in Structure? (Y/N)

Y

fseat

0.8

flink

0.8

fskew

1.422612

f

0.910471

Vmj

8.194243

Support Overlap Length Known (Y/N)

N

Overlap Length (mm)

#####

Bridge Constructed Before 1973? (Y/N)

N

Note: Bridge may have been constructed pre-1973, but bridge was redecked in 1991. Assume that any seating length would have been brought up to current standards, and linkage bolts fitted.

Length of Bridge (m)

19

Horizontal Earthquake Linkage Bolts?

(Y/N)

Y

Skew Angle

12.5

PC Girder Bridge with Full Width Diaphragm Movement at Joints? (Y/N)

Y

Column/Wall Vulnerability Rating

Is there More than 1 span?

(Y/N)

Y

Vcw

5

R/C Column Present? (Y/N)

N

Designed Prior to 1973? (Y/N)

#####

Foundation/Abutments Vulnerability Rating

Erosion/Scour Factor

1.5

feros

1.5

Vfa

6

Is there more than one span?

Y

Is the Bridge length >5m

Y

Vulnerability Rating of the Structure

Soil

Factor

1.5

Vstruc

8.194243

Vulnerability	V
MANGATAWAI STREAM BRIDGE	8.4

OTURERE STREAM BRIDGE

Movement Joint Vulnerability Rating
 Movement Joints in Structure? (Y/N) Y
 fseat 0.466642 flink 0.8 fskew 1 f 0.373313 Vmj 3.733134

Support Overlap Length Known (Y/N)
 Y

Overlap Length (mm) 500
 Bridge Constructed Before 1973? (Y/N) #####

Length of Bridge (m)
 11.9

Horizontal Earthquake Linkage Bolts? (Y/N)
 Y

Skew Angle
 0

PC Girder Bridge with Full Width Diaphragm Movement at Joints? (Y/N)
 N

Column/Wall Vulnerability Rating
 Is there More than 1 span? (Y/N) N
 Vcw 0

R/C Column Present? (Y/N)
 #####

Designed Prior to 1973? (Y/N)
 #####

Foundation/Abutments Vulnerability Rating
 Erosion/Scour Factor 1.5
 feros 1.5 Vfa 4.5

Is there more than one span?
 N

Is the Bridge length >5m
 Y

Vulnerability Rating of the Structure
 Soil Factor 1.5
 Vstruc 4.5

Vulnerability	V
OTURERE STREAM BRIDGE	5.1

WAIHOHONU STREAM BRIDGE

Movement Joint Vulnerability Rating	fseat	flink	fskew	f	Vmj
Movement Joints in Structure? (Y/N)	0.6499	1	1	0.6499	6.4998
Y					

Support Overlap Length Known (Y/N)

Y

Overlap Length (mm)	Bridge Constructed Before 1973? (Y/N)
450	N

Length of Bridge (m)

33.33

Horizontal Earthquake Linkage Bolts? (Y/N)

N

Skew Angle

0

PC Girder Bridge with Full Width Diaphragm Movement at Joints? (Y/N)

N

Column/Wall Vulnerability Rating	Vcw
Is there More than 1 span? (Y/N)	10
Y	

R/C Column Present? (Y/N)

Y

Designed Prior to 1973? (Y/N)

Y

Foundation/Abutments Vulnerability Rating	feros	Vfa
Erosion/Scour Factor	1.5	6
1.5		

Is there more than one span?

Y

Is the Bridge length >5m

Y

Vulnerability Rating of the Structure	Vstruc
Soil Factor	10
1.5	

Vulnerability	V
WAIHOHONU STREAM BRIDGE	10.0

MANGATOETOENUI STREAM BRIDGE

Movement Joint Vulnerability Rating	fseat	flink	fskew	f	Vmj
Movement Joints in Structure? (Y/N)	0.589043	1	1	0.589043	5.89043
Y					
Support Overlap Length Known (Y/N)					
Y					
Overlap Length (mm)					Bridge Constructed Before 1973? (Y/N)
450					####
Length of Bridge (m)					Note: Overlap length measured of drawings. No details of linkage bolts.
18.3					
Horizontal Earthquake Linkage Bolts? (Y/N)					
N					
Skew Angle					
0					
PC Girder Bridge with Full Width Diaphragm Movement at Joints? (Y/N)					
N					
Column/Wall Vulnerability Rating					Vcw
Is there More than 1 span? (Y/N)					0
N					
R/C Column Present? (Y/N)					
####					
Designed Prior to 1973? (Y/N)					
####					
Foundation/Abutments Vulnerability Rating				feros	Vfa
Erosion/Scour Factor				1.5	4.5
1.5					
Is there more than one span?					
N					
Is the Bridge length >5m					
Y					
Vulnerability Rating of the Structure					Vstruc
Soil Factor					5.890431
1.5					

Vulnerability	V
MANGATOETOENUI STREAM BRIDGE	6.4

WAIKATO RIVER BRIDGE

Movement Joint Vulnerability Rating	fseat	flink	fskew	f	Vmj
Movement Joints in Structure? (Y/N)	0.653731	0.8	1	0.522985	4.706866

Y

Support Overlap Length Known (Y/N)

Y

Overlap Length (mm)

400

Bridge Constructed Before 1973?
(Y/N)

####

Length of Bridge (m)

12

Horizontal Earthquake Linkage Bolts? (Y/N)

Y

Skew Angle

0

PC Girder Bridge with Full Width Diaphragm Movement at Joints? (Y/N)

Y

Column/Wall Vulnerability Rating

Vcw

Is there More than 1 span? (Y/N)

0

N

R/C Column Present? (Y/N)

####

Designed Prior to 1973? (Y/N)

####

Foundation/Abutments Vulnerability Rating

feros

Vfa

Erosion/Scour Factor

1.5

4.5

1.5

Is there more than one span?

N

Is the Bridge length >5m

Y

Vulnerability Rating of the Structure

Vstruc

Soil Factor

4.706866

1.5

Vulnerability	V
WAIKATO RIVER BRIDGE	5.3

APPENDIX E

Vulnerability Rating Of Bridges Along Alternative Routes

SH 4 -National Park to Raetihi				
Bridge Name	V_{MJ}	V_{CW}	V_{FA}	Vulnerability Rating (V)
Mangaturuturu Stream	6.4	5	6	6.9
Manganui-A-Te-Ao River	10*	5	6	10.0
Makatote Stream	5.1	10	6	10.0
Makomiko Stream	4.6	0	4.5	5.2
Waimarino Overbridge	4.2	5	6	6.5
Waimarino Stream	4.7	0	4.5	5.2
Makara Stream	6.2	5	6	6.6

SH 47				
Bridge Name	V_{MJ}	V_{CW}	V_{FA}	Vulnerability Rating (V)
Te Whaiiau Stream	4.6	0	4.5	5.2
Wanganui River (Headwaters)	5.3	0	4.5	5.9
Mangatepopo Stream	5.7	0	4.5	6.2
Papamanuka Stream No 1	4.6	0	4.5	5.2
Whakapapanui Stream	5.9	0	4.5	6.4
Whakapapaiti Stream	6	10	6	10.0
Mangahuia Stream	5.9	10	6	10.0
Wairehu Canal	9.4	0	4.5	9.5

SH 49				
Bridge Name	V_{MJ}	V_{CW}	V_{FA}	Vulnerability Rating (V)
Waitangi Stream	10*	0	4.5	10.0
Whangaehu River	10*	5	6	10.0
Tokiahuru Stream	6.3	5	6	6.7
Waitaki Stream	4.3	5	6	6.5
Waiharuru Stream	5.8	0	4.5	6.3
Mangaehuehu Stream	9*	0	4.5	9.1
Mangateitei Clyde St	3.9	5	6	6.5
Mangawhero River	5.8	5	6	6.5
Hapuwhenua Stream	9*	0	4.5	9.1
Makotuku Stream	3.7	10	6	10.0

SH 41				
Bridge Name	V _{MJ}	V _{CW}	V _{FA}	Vulnerability Rating (V)
Tokaanu Tailrace	0	0	4.5	5.1
Tokaanu Stream	7	5	6	7.3
Waihi Stream	10*	10	6	10.0
Lower Kuratau River	0	5	6	6.5
Upper Kuratau River	0	5	6	6.5

* The overlap length was unknown. This meant that the Movement Joint Vulnerability was determined using only the year of construction of the bridge. Further investigation as to the actual seating lengths on the bridges may reduce their calculated vulnerability's.

APPENDIX F

Schematic Representation of the Buffer Network Used to Model the Central North Island.

APPENDIX G

Proposed Waikato Stream to Wharepu Realignment

APPENDIX H

User Interface for Risk Optimisation Software

Control Worksheet	
Expenditure modes	Failure modes
<input type="text" value="4"/>	<input type="text" value="3"/>
Title	
<input type="text" value="Road Closure Mitigation"/>	
Step Size (\$)	Calculation status
<input type="text" value="1"/>	<input type="text" value="100%"/>
Gap between records (number of steps)	
<input type="text" value="50"/>	
Total expenditure (\$)	
<input type="text" value="1000"/>	
Expenditure to reduce the probability of failure? Y/N	
<input type="text" value="N"/>	
Expenditure to reduce the cost of failure? Y/N	
<input type="text" value="Y"/>	
Check the details in the Yellow cells, and change as required.	
To move to the next stage click here>>	
<input type="button" value="NEXT>>"/>	

Expenditure Combinations Table

Enter the Relationship types and then press>>

NEXT>>

Check that the relationships are correct, if not, redo above step.
If relationships are correct, then move to next stage>>

NEXT>>

Apply Salt	Apply CMA	Purchase Grader	Retrofit Bridge	
	1 <<Relationship Type Mutually Exclusive Expenditure Modes	0 <<Relationship Type Independant Expenditure Modes	0 <<Relationship Type Independant Expenditure Modes	Apply Salt
1 <<Relationship Type Mutually Exclusive Expenditure Modes		0 <<Relationship Type Independant Expenditure Modes	0 <<Relationship Type Independant Expenditure Modes	Apply CMA
0 <<Relationship Type Independant Expenditure Modes	0 <<Relationship Type Independant Expenditure Modes		0 <<Relationship Type Independant Expenditure Modes	Purchase Grader
0 <<Relationship Type Independant Expenditure Modes	0 <<Relationship Type Independant Expenditure Modes	0 <<Relationship Type Independant Expenditure Modes		Retrofit Bridge

0: <INDEPENDANT EXPENDITURE MODES

Expenditure in area A has NO effect on the possibility of expenditure in area B.

1: <MUTUALLY EXCLUSIVE EXPENDITURE MODES

IF there is expenditure in area A, THEN there cannot be expenditure in area B, and vice versa.

2: <DEPENDENT EXPENDITURE MODES

IF there is expenditure in area B, THEN there may also be expenditure in area A.

NOTE: Expenditure in area A denotes the expenditure modes along the top of the table, and area B, along the end of the table.

?

Risk Reduction Relationships

Enter the Relationship types (the Yellow cells) and then press>>

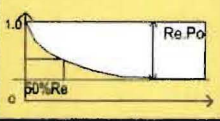
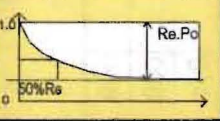

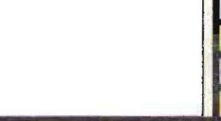




NEXT>>

Check that the relationships are correct, if not, redo above step.

Fill in all details required (the Yellow cells).

If relationships and details are correct, then move to next stage>>

NEXT>>

Apply Salt		Apply CMA		Purchase Grader		Retrofit Bridge		
2	<< Graph Type	2	<< Graph Type	4	STEP FUNCTION	4	STEP FUNCTION	Snow and Ice
50 % Re.		50 % Re.		Exp. Step		Exp. Step		Probability
300		500		150		150		0.5
Re. Po.		Re. Po.		Effectiveness		Effectiveness		Cost
0.65		0.95		0.15		0		100,000
0	<< Graph Type	0	<< Graph Type	4	STEP FUNCTION	4	STEP FUNCTION	Earthquakes
No relationship between expenditure and failure mode		No relationship between expenditure and failure mode		Exp. Step		Exp. Step		Probability
				150		150		0.1
				Effectiveness		Effectiveness		Cost
				0		0.7		50,000
0	<< Graph Type	0	<< Graph Type	4	STEP FUNCTION	4	STEP FUNCTION	Volcanic Eruption
No relationship between expenditure and failure mode		No relationship between expenditure and failure mode		Exp. Step		Exp. Step		Probability
				150		150		0.01
				Effectiveness		Effectiveness		Cost
				0.1		0.15		10,000

0: <NO RELATIONSHIP

Expenditure in this area has no effect on the probability of this type of failure.

1: <CURTAILED POWER RELATIONSHIP

Expenditure in this area reduces the probability of this event to zero, in a power relationship.

2: <LIMITED EXPONENTIAL RELATIONSHIP

Expenditure in this area will reduce failure probabilities exponentially, with a limit to the amount the probability may be reduced.

3: <USER DEFINED FUNCTION

The user may specify the way in which probabilities are reduced with expenditure.

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Optimum Spending Portfolios

Road Closure Mitigation

Expend.	Risk	Apply Salt	Apply CMA	Purchase Grader	Retrofit Bridge
0.0	55100.00	0	0	0	0
50.0	51621.17	50	0	0	0
100.0	48454.91	100	0	0	0
150.0	45634.10	150	0	0	0
200.0	43121.04	200	0	0	0
250.0	40882.16	250	0	0	0
300.0	38887.54	300	0	0	0
350.0	36880.19	0	350	0	0
400.0	34919.37	0	400	0	0
450.0	33089.87	0	450	0	0
500.0	31382.88	0	500	0	0
550.0	29790.21	0	550	0	0
600.0	28304.19	0	600	0	0
650.0	26917.69	0	650	0	0
700.0	25624.03	0	700	0	0
750.0	24417.01	0	750	0	0
800.0	23290.82	0	800	0	0
850.0	22109.03	0	700	0	150
900.0	20902.01	0	750	0	150
950.0	19775.82	0	800	0	150
1000.0	18725.05	0	850	0	150

**** Note:** These results are intended as an example only, to illustrate to the reader the type of results that are obtainable. They do not reflect the findings of this report.